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URBAN STORMWATER MANAGEMENT MODELING AND DECISION-MAKING

FLORIDA UNIVERSITY

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The purposes of this study were to test, refine and augment the capabilities of the EPA Storm Water Management Model (SWMM), and to develop decision-making capabilities for use in the study of urban stormwater runoff problems.

With regard to the SWMM the numerous programming refinements were introduced to provide greater reliability and flexibility. A sediment prediction capability has been incorporated directly into the SWMM. Detailed testing was conducted in Lancaster, Pennsylvania to demonstrate the ability of the SWMM to describe the relatively complex phenomena occurring in an urban catchment.

A systematic procedure is presented for examining the stormwater management problem in the broader context of urban water resources management. Related standards for flood control and drainage, street and parking lot design, etc., are reviewed and suggestions presented regarding modifications in practices which would ameliorate stormwater problems. An optimization procedure is described which addresses the related problems of finding efficient and equitable control strategies. Lastly, the use of the SWMM for preliminary hydraulic design of sewer systems is described.

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MODELING AND DECISION-MAKING

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FOREWORD

Man and his environment must be protected from the adverse effects of pesticides, radiation, noise and other forms of pollution, and the unwise management of solid waste. Efforts to protect the environment require a focus that recognizes the interplay between the components of our physical environment--air, water, and land. The National Environmental Research Centers provide this multidisciplinary focus through programs engaged in

- studies on the effects of environmental contaminants on man and the biosphere, and
- a search for ways to prevent contamination and to recycle valuable resources.

This study tests, refines, and augments the capabilities of the EPA Storm Water Management Model (SWMM). Also, decision-making capabilities are developed for use in the study of urban storm water runoff problems. The storm water management problem is viewed in the broader context of urban water resources management. The results indicate numerous areas where more comprehensive environmental management strategies could be effectively applied.

A. W. Breidenbach, Ph.D.
Director
National Environmental
Research Center, Cincinnati

ABSTRACT

The purposes of this study were to test, refine and augment the capabilities of the EPA Storm Water Management Model (SWMM), and to develop decision-making capabilities for use in the study of urban storm water runoff problems.

With regard to the SWMM the numerous programming refinements were introduced to provide greater reliability and flexibility. A sediment prediction capability has been incorporated directly into the SWMM. Detailed testing was conducted in Lancaster, Pennsylvania to demonstrate the ability of the SWMM to describe the relatively complex phenomena occurring in an urban catchment.

A systematic procedure is presented for examining the storm water management problem in the broader context of urban water resources management. Related standards for flood control and drainage, street and parking lot design, etc. are reviewed and suggestions presented regarding modifications in practices which would ameliorate storm water problems. An optimization procedure is described which addresses the related problems of finding efficient and equitable control strategies. Linear programming techniques are combined with procedures from cooperative N-person game theory to examine these questions.

Lastly, the use of the SWMM for preliminary hydraulic design of sewer systems is described. The runoff portion provides a more accurate inlet hydrograph and the transport portion is used for refined hydraulic analysis of phenomena such as surcharging.

This report was submitted in fulfillment of Grant No. 802219 (11023 GSC) by the Department of Environmental Engineering Sciences, University of Florida, under the sponsorship of the U.S. Environmental Protection Agency. Work was completed as of June 1973.

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The SWMM is a large and complex model which requires a significant commitment of resources in order to use it properly. We are fortunate to have a systems analyst, Mr. W. Alan Peltz, who has devoted a considerable amount of his time debugging, simplifying and modifying the model so that its usefulness will be improved. We are indebted to Ms. Polinski for her patience and persistence in typing numerous drafts of this report. Mr. Damon Hooten and Ms. Gina Ellis conscientiously drafted figures and diagrams.

Computations were performed on the IBM 370/165 at the Northeast Regional Data Center at the University of Florida.

SECTION I

CONCLUSIONS

Overall project goals were to test, refine and augment the capabilities of the EPA Storm Water Management Model (SWMM), and to develop decision-making techniques for use in the study of urban runoff problems. This report represents the findings of the University of Florida with respect to the specific objectives outlined below:

Objectives, Task I:

1) test present SWMM using data from Lancaster, Pennsylvania;
2) provide programming improvements to SWMM;
3) provide model refinements to SWMM; and
4) compile alternative methods of pollution control and associated cost data.

Objectives, Task II:

1) provide detailed cost functions for pollution control options;
2) develop evaluation and monitoring routine for assessment of pollution significance;
3) develop decision-making methodology for least cost abatement of pollution and drainage problems and provide necessary interfaces in SWMM; and
4) develop hydraulic design capability for the SWMM.

Findings are presented in Section IV - Refinements of the SWMM, Section V - Testing of SWMM in Lancaster, Pennsylvania, Section VI - Decision-Making for Water Quantity and Quality Control, and Section VII - Hydraulic Design by the SWMM.

With regard to Section IV, numerous programming refinements were introduced to provide greater reliability and flexibility in using the Model. Field studies in Lancaster, Pennsylvania and Gainesville, Florida indicated the importance of soil and vegetation as sources of stormwater pollution. Thus, special studies were conducted to obtain a better appreciation of these phenomena. As a result of these studies, a sediment prediction capability has been incorporated directly into the SWMM. This method uses the Universal Soil Loss Equation which was originally developed for agricultural areas. The amount of tree and grass litter in urban areas can be estimated using the results presented in a latter part of Section IV. This review indicates that improperly managed collection and disposal systems for these materials can cause significant deleterious effects on the receiving water.

Refinements were made in the Treatment Model to reflect recent developments such as swirl concentrators. Similarly, refinements in the Receiving Model are presented which further increase the flexibility of this Model.

Also included in Section IV is a procedure for ranking the nodes based on their selective importance as contributors of pollution. Furthermore, the magnitude of the source is partitioned into its subcomponents.

The testing in Lancaster presented in Section V illustrates the ability of the Model to describe the relatively complex phenomena occurring in an urban catchment. Unfortunately, only very limited and somewhat questionable quantity and quality verification data were available. As a result, agreement between predicted and measured hydrographs and pollutographs was only fair. Quality predictions in the Conestoga River, the receiving waterway, were reasonably close to the limited measured data available. The analysis indicates an improved capability to describe the various processes but also indicates the need for a better data base to properly calibrate the model.

The Decision-Making Model for urban stormwater management is presented in Section VI. A systematic procedure is presented for examining the problem in the broader context of urban water resources management. The inventory of sources of pollutants includes reference to related models for air pollution control. Examination of the effect of urbanization on stormwater runoff indicates that significantly higher peak flows accompany urbanization. Thus, control costs increase substantially at higher levels of urbanization.

Studies of parking practices and residential street design indicate there has been a significant increase in paved area per household in order to accommodate an automobile-oriented society. Residential areas are the most significant source of storm water since they are the predominant land use. It appears that significant reductions in the amount of paved area could be realized with little loss in value for residents. Residential streets could be made narrower in recognition of the availability of off-street parking. Because of their low rate of utilization, it appears feasible to temporarily pond water on many residential streets. Curb and gutter could be eliminated in some areas which would substantially reduce road construction costs and achieve on-site control of storm water. Inlets could be designed to incorporate grates and prevent "trash" from entering the storm sewer. Current inlet design optimizes hydraulic efficiency to the detriment of the receiving water.

Current standards for parking lot design provide a very high level of service, *e.g.*, shopping centers are designed with adequate parking for all but the peak 10 hours of the year. They are utilized at a much lower rate during the balance of the year. It appears feasible to temporarily pond water on parking lots and/or redesign them to include pervious areas to permit percolation. The use of compact cars would reduce the paved area by about one third.

In summary, it appears that the impervious areas associated with the automobile are the most important source of the stormwater problems. These developments have resulted in a significant increase in paved area. By contrast, more innovative urban planning concepts are reversing this trend by utilizing paved area much more efficiently than in the past. The economic analysis presents a revised and expanded list of cost functions for the various control options.

Following a review of alternative performance standards for stormwater quality management, we have recommended that the control system should provide acceptable performance for the "minor storm" as defined in the Drainage Criteria Manual. To a large extent, stormwater quantity and quality management are complementary. The suggested performance criteria are:

1) no increase in the peak flow observed prior to development; and

2) adequate treatment of storm water prior to release to the receiving environment.

These standards can be satisfied on-site or off-site by forming a coalition with others for the same purpose.

In any given city, there are a wide variety of control alternatives to be considered. Thus, the Optimization Model was set up in a general manner. The emphasis was on describing principles for efficient and equitable control strategies. A procedure is included for placing a value on off-site disposal based on alternative on-site control costs. Much research has been done in determining the best *overall* solution. However, little effort has been made to examine whether this solution is fair to everyone. Using notions from cooperative N-person game theory, procedures are described for solving this important equity question. Unfortunately, the scope of the research did not allow a detailed example of the methodology.

The purpose of the Decision-Making Model for urban stormwater quantity and quality management is to provide a systematic and comprehensive procedure for solving this important problem. A review of the technical aspects of the problem indicates that stormwater quantity and quality problems seem to grow more serious as urbanization continues. Consequently, even using sophisticated engineering controls, it does not seem possible to devise a control strategy which will have long-term value. These observations led to a questioning of the wisdom of this open-ended approach.

The proposed quasi-closed system approach solves the above problems by internalizing the responsibility for stormwater control so that those people causing the problem are assigned responsibility for adequate collection and safe disposition of this water. Ideally, this principle could be applied without discrimination. With this policy, at low levels of urbanization there is usually a relatively large amount of open space such as flood plains, etc., which would provide a "natural" solution to the problem. Thus, control costs would be low. However, as the area reaches a higher level of urbanization, many of these natural areas are either being fully utilized or have been developed. Thus, additional development needs to acquire their own control facilities. Where stormwater quantity and quality constraints are placed on a new development, it appears to result in the type of development which is very much in vogue in current urban planning.

Section VII shows how the SWMM can be used for hydraulic design of sewer systems. The Runoff portion of the SWMM permits one to generate a more accurate inlet hydrograph. Transport permits a much improved characterization of the hydraulics of flow through sewers. For a specified system configuration, a routine has been added which checks for surcharge conditions and, if present, enlarges the downstream conduit by standard amounts until capacity exists to accept the flow. An example illustrates the useful ability of an undersized system to store excess flows and thus reduce the peaks of downstream hydrographs.

SECTION II

RECOMMENDATIONS

Relatively sophisticated mathematical models such as the SWMM should be quite appealing to environmental decision-makers as a management tool. However, the state of the art in mathematical simulation models has reached a level of efficiency that is not being matched by the data-gathering agencies. Invariably, a successful application of the SWMM requires:

1) a considerable amount of input data ranging from physical watershed information to cost functions for selection of the most practical treatment alternatives;

2) frequent field measurements of real storm events for model running and calibration procedures; and

3) that the data-gathering efforts to satisfy requirements (1) and (2) should be established on as permanent a basis as possible to enable the SWMM user to continuously update the data base. The following recommendations are made for furthering the continued use of the SWMM:

    a) Municipalities and other governmental agencies concerned with stormwater management should be encouraged to gather the type of information (such as erosion) which is vital to a complete, meaningful and frequent application of management tools such as SWMM in the interest of reducing the overall impact of storm water on the urban community.

    b) It is recommended that the use of the SWMM for analyzing real systems with real problems be continued and expanded. Many refinements and improvements have resulted from such interactions.

    c) It is recommended that more explicit considerations be given to the interrelationships between urban stormwater quality management and urban water resources management. The SWMM appears to be the most sophisticated model for analyzing these broader questions.

d) Finally, it is recommended that the appropriate professional organizations be encouraged to re-examine the wisdom of existing practices in residential street design, parking lot design, and urban drainage design. Design practices for these functions appear to have evolved with little consideration for their overall impact on the urban area.

SECTION III

INTRODUCTION

A. COMBINED AND STORM SEWER OVERFLOWS

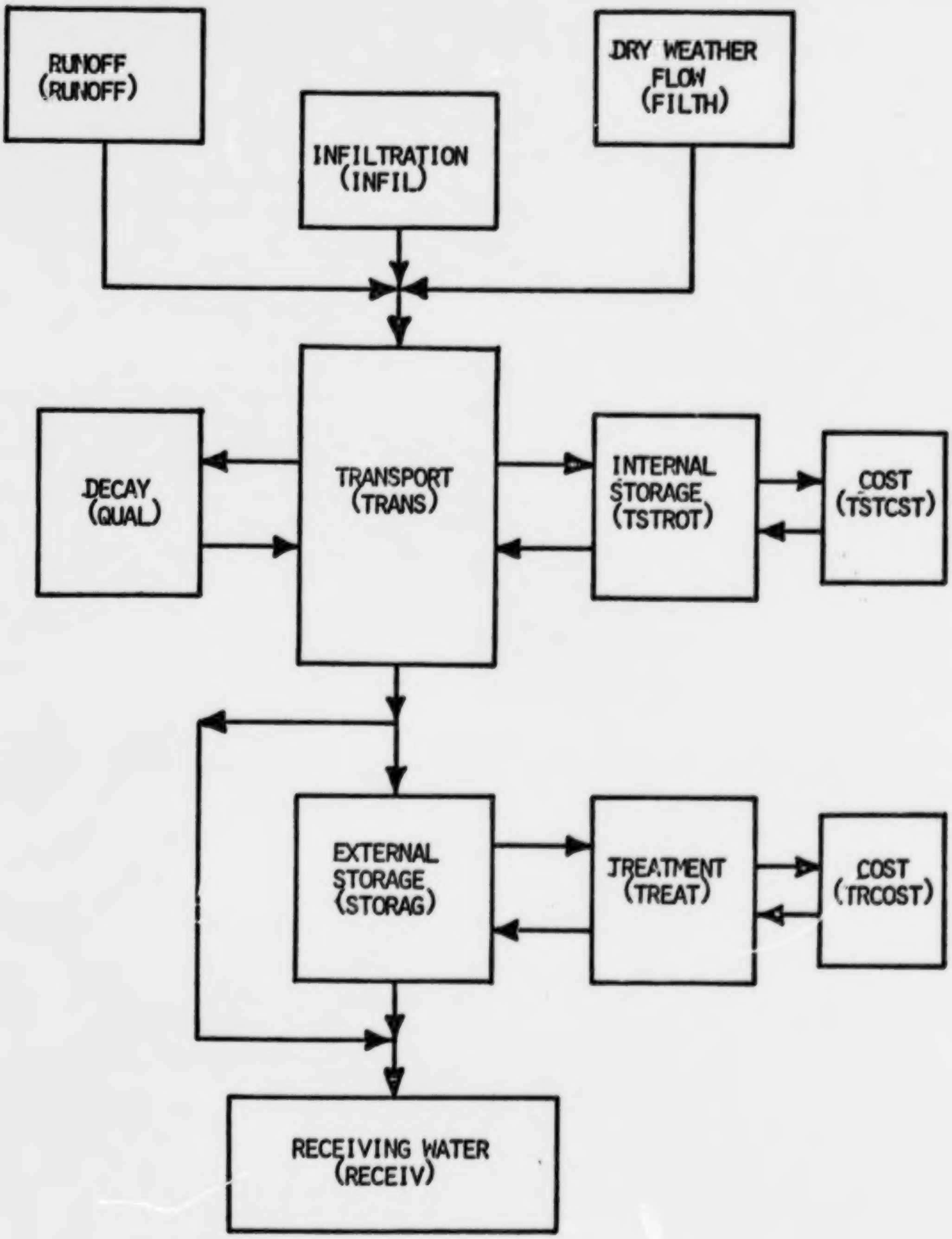
An enormous pollution load is placed on streams and other receiving waters by combined and separate storm sewer overflows. It has been estimated that the total pounds of pollutants (BOD and suspended solids) contributed yearly to receiving waters by such overflows is of the same order of magnitude as that released by all secondary sewage treatment facilities (Gameson and Davidson, 1964; Field and Struzeski, 1972). The Environmental Protection Agency (EPA) has recognized this problem and led and coordinated efforts to develop and demonstrate pollution abatement procedures (Field and Struzeski, 1972). These procedures include not only improved treatment and storage facilities, but also possibilities for upstream abatement alternatives such as rooftop and parking lot retention, increased soil infiltration, improved street sweeping, retention basins and catch-basin cleaning or removal. The complexities and costs of proposed abatement procedures require that care and effort be expended by municipalities and others charged with decision-making for the solution of these problems.

B. STORM WATER MANAGEMENT MODEL

It was recognized that an invaluable tool to decision makers would be a comprehensive mathematical computer simulation program that would accurately model quantity (flow) and quality (concentration), during the total urban rainfall-runoff process. This model would not only provide an accurate representation of the physical system, but also provide an opportunity to determine the effect of proposed pollution abatement procedures. Alternatives could then be tested on the model and least cost solutions could be developed.

As a result, the University of Florida (UF), Metcalf and Eddy, Inc., Engineers (ME) and Water Resources Engineers (WRE) were awarded a joint contract for the development, demonstration and verification of the Storm Water Management Model (SWMM). The resulting model, completed in October, 1970, has been documented (EPA, 1971 a,b,c,d), and is presently being used by a variety of consulting firms and universities.

The present SWMM is descriptive in nature and will model most urban configurations and encompasses rainfall, runoff, drainage, storage-treatment, and receiving waters. The major components of the SWMM are illustrated in Figure 1. However, it neither defines nor determines any decisions for the system, nor does it consider alternative methods for efficient economic comparisons.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DRY WEATHER
FLOW
(FILTH)
DECAY
(QUAL)
TRANSPORT
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STORAGE
(TSTROT)
COST
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EXTERNAL
STORAGE
(STORAG)
TREATMENT
(TREAT)
COST
(TRCOST)
RECEIVING WATER
(RECEIV)

FIGURE 1.
Overview of the Model Structure.
Subroutine names are shown in parentheses.

C. DECISION-MAKING

In recognition of the need for improved decision-making capabilities, the University of Florida submitted a proposal to EPA titled "Decision-Making Model for the Management of Storm Water Pollution Control" in which it was intended to provide a systematic procedure which could be applied to a wide variety of specific circumstances in support of intelligent management decisions. The work required to obtain a least cost solution would be considerably reduced by means of determining the origin of the most severe pollution load, consideration of all upstream and downstream pollution abatement procedures and associated costs, and through the possible use of mathematical optimization techniques.

The project was funded as part of an EPA Demonstration Grant to Lancaster, Pennsylvania (EPA No. 802219 (formerly 11023 GSC), in which an underground "silo," a swirl concentrator and a microstrainer were to be installed at the outfall of the Stevens Avenue Drainage District to control overflow into the Conestoga River (full details are given in Chapter V). This report represents the findings of the University of Florida with respect to specific objectives as defined below:

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1) test present SWMM using data from Lancaster, Pennsylvania;
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3) develop decision-making methodology for least cost abatement of pollution and drainage problems and provide necessary interfaces in SWMM; and
4) develop a hydraulic design capability for the SWMM.

Because of funding problems, this report does not cover Task III of the research, as included in the original proposal, which is a demonstration of the improved SWMM and decision-making model in Lancaster. Thus, a detailed application of new techniques discussed herein is not included. Rather, the emphasis is on the results of research to date. Model improvements, refinements and testing are discussed in Section V and the hydraulic design capability is illustrated in Section VII. The rest of the objectives including details of costs, monitoring and the decision-making methodology are discussed in Section VI.

SECTION IV

REFINEMENTS OF THE SWMM

A. PROGRAMMING IMPROVEMENTS

Utilization of the Storm Water Management Model has resulted in an extensive array of alterations, ranging from remedies for programming errors to inclusion of new routines. Major refinements are discussed in the next section, while general improvements are considered below.

Most large computer programs have their share of "bugs" that appear from time to time during usage, and the SWMM is no exception. Inasmuch as the model has been extensively altered since its original formulation, simple corrections to the original program are not easily identified and listed. Rather, users are advised to obtain the most recent version from EPA, which will include all corrections and refinements to date.

Input/output data-sets have been made compatible among all programs, such that it is possible for output from a Transport run to serve as input to another Transport run, for example. This enables large or complicated networks to be simulated in a sequential fashion. However, a common time step is required for all models, except for input to Receiving.

In the original Transport Model, quality routing was performed incorrectly during surcharge conditions. This has been corrected, and the revisions may be seen in a listing of the current UF version of the SWMM.

Refinements to several routines (see next section) have necessitated some changes in program input requirements. Release 2 of the EPA Storm Water Management Model is an improved version of the original model that retains all of the original model's features, as well as the refinements to be discussed subsequently. New options are used only upon request of the user, and the default condition omits them.

B. EVALUATION AND PREDICTION OF URBAN SEDIMENT YIELD-OVERVIEW

It is estimated that naturally vegetated land areas erode at an average of 115 tons/mile^2/year (4.08 x 10^4 kg/km^2/yr) (Ports, 1973; Brandt, 1972). Of course, this figure varies considerably at different locations, depending upon such things as climate, soil, terrain, and vegetative cover. For example, natural areas of the Potomac

River Basin have sediment yields of 15 to 20 tons/mile^2/year (5.33 to 7.10 kg/km^2/yr) as compared with yields greater than 320 tons/mile^2/year (1.14 x 10^5 kg/km^2/yr) throughout geological history for the Mississippi River Basin (Brandt, 1972). Natural erosion and sedimentation are vital geological processes by which nutrients travel from the land to the oceans. Considerations for controlling these processes should be tempered with an understanding of the natural system. Evaluation of alternatives to control excessive erosion should be undertaken on a river basin or regional basis where both natural and man-induced sediment yields can be measured or predicted.

Today, man has certainly increased the rate of erosion and the consequent siltation and sedimentation of rivers in areas of his most intense activity. The increase is a result of activities which remove natural ground cover, thus allowing immediate soil-water contact and increased impermeability. This increased impermeability causes runoff of greater velocity which more readily transports the available soil. As an example of the effect of man's actions, estimates for the Potomac River Basin indicate that the Washington DC area produces nearly one third of the total sediment discharged into the estuary. The Washington DC area encompasses less than 5% of the watershed (Brandt, 1972). In the northern Mississippi River Basin, studies indicate an increase in erosion occurs as the land use goes from forested watershed to pasture land to cultivated land use (Brandt, 1972). A summary of these and other pertinent data relating erosion rates to land use is given in Table 1 (Brandt, 1972).

Sediment Production

Erosion from construction sites in and around the urban area is probably the greatest source of sediment yield per unit land area. Estimates of sediment yield for urban areas undergoing construction range from 280 to 100,000 tons/mile^2/year (3.55 x 10^7 kg/km^2/yr) (see Table 1). Sediment yields from single construction sites may vary from 2 to 200 times as much as from naturally vegetated areas (Brandt, 1972). In Table 2, crop management factors (C factors) which are used in predicting gross annual erosion by the Universal Soil Loss Equation are listed (Brandt, 1972). The erosion prediction increases linearly with increasing value of C. These prediction factors indicate that erosion due to construction is on the order of 100 times greater than completely developed urban sites, all else being equal.

In addition to localized soil erosion from construction sites, open land, and stream banks, sediment is also found rather uniformly throughout the urban area. This sediment is often referred to as dust and dirt and the total annual yield can be partly attributed to localized soil erosion, but probably a greater part is made up of

Table 1.
EROSION RATES REPORTED FOR VARIOUS SEDIMENT SOURCES
(Brandt, 1972)

Sediment source	Erosion rate (ton/mile^2/year)	Geographic location	Comment
Natural	15-20	Potomac River Basin	Native cover
	32-192		Native cover
	200	Pennsylvania and Virginia	Natural drainage basin
	320	Mississippi River Basin	Throughout geologic history
	13-83	Northern Mississippi	Forested watershed
	25-100	Northwest New Jersey	Forest and underdeveloped land
	115		Soils eroding at the rate they form
Agricultural	12,800	Missouri Valley	Loess region
	13,900	Northern Mississippi	Cultivated land
	1,030	Northern Mississippi	Pasture land
	10,000-70,000		Continuous row crop without conservation practices
	200-500	Eastern U. S. Piedmont	Farmland
	320-3,840		Established as tolerable erosion
Urban	50,000	Kensington, Maryland	Undergoing extensive construction
	1,000-100,000		Small urban construction area
	1,000	Washington, D. C. area	750 mile^2 area average
	500	Philadelphia area	
	146	Washington, D. C. area watersheds	As urbanization increases
	280		
	690		
	2,300		
Highway construction	36,000	Fairfax County, Virginia	Construction on 179 acs.
	50,000-150,000	Georgia	Cut slopes

Table 2. FACTOR C VALUES USED IN COMPUTING GROSS EROSION (Brandt, 1972)

Land Use	Factor C Value
Cropland	0.08
Grassland	0.01
Woodland	0.005
Construction	1.0
Urban	0.01

air pollution dust fallout, building and demolition wastes, eroded pavement, and less discernible particulate accumulation due to wind, traffic, and human activity. These materials are similar to natural sediment in size, shape and composition, and therefore should be considered as part of the total urban sediment load.

In a study made in Chicago, the average dust and dirt yield was found to be 79 lbs/curb-mile/day (22.3 kg/curb-km/day) or 14 tons/curb-mile/year (7.9 MT/curb-km/yr) (American Public Works Association, 1969). Since these data were collected in flat developed areas of varying land use, the contribution due to heavy soil erosion from construction sites and vacant lots can be considered negligible. A similar study on eight US cities indicated an average total solids loading rate of 348 lbs/curb-mile/day (98 kg/curb-km/day) (Sartor and Gail, 1972). The Chicago study indicated that 75 percent of the total solids was made up of dust and dirt having particle size less than 3200 microns. In the second study, 75 percent of the total solids had a particle size of less than 2000 microns and can be considered dust and dirt. The eight-city study then indicates an average street sediment loading rate of 261 lbs/curb-mile/day (73.53 kg/curb-km/day). Table 3 summarizes these data for both studies.

A typical Chicago residential area has a ratio of curb length to land area of 1 mile to .0235 miles^2 or 42.5 mile^{-1} (26.4 km^{-1}). This figure can be used to convert dust and dirt data for residential Chicago to lbs/mile^2/year for comparison with erosion data from urban areas. Thus Chicago street sediment yield is 287 and 940 tons/mile^2/year (1.02×10^5 and 3.35×10^5 kg/km^2/yr) for single and multiple family residential areas respectively. As seen from Table 1 , these dust and dirt estimates are similar to erosion data from the Washington DC area.

Sediment Control Alternatives

The distinctions made between erosion and street dust and dirt as sources of urban sediment are very useful in considering sediment control alternatives. Erosion from exposed soil surfaces can best be controlled at the site of erosion by various methods of applying cover to hold the soil and/or by using sediment basins or control structures to collect the soil from the runoff. In the case of construction site erosion, development practices also influence the extent of erosion. Street sediment on the other hand, can be controlled by street sweeping, catchbasin cleaning, or by collection through retention of stormwater at the downstream end of the drainage network. In order to evaluate the effectiveness of these general control alternatives, estimates of the magnitude of sediment yielded from each type of source must be made for both before and after conditions.

Table 3. COMPARISON OF DATA FROM TWO STUDIES ON DUST AND DIRT COLLECTED IN STREETS
(lb/curb mile/day)

	Chicago(1)	Eight cities(2)	
Land use	Dust & dirt	Total solids	Dust & dirt = .75 total solids
Commercial	174	226	170
Industrial	243	447	335
Residential		373	280
Multi-family	121		
Single family	37		
Weighted average	79	348	261

(1) APWA, 1969

(2) Sartor and Gail, 1972

Soil Detachment by Rainfall

Raindrop impact has been studied by C. K. Mutchler and R. A. Young (1972) both at USDA Sedimentation Laboratory, to establish the direct role of raindrops in the mechanics of erosion. Raindrop impact is the primary source of energy for detaching soil from any land area not protected by cover of some type. It also plays an important part in sediment transport by overland flow (runoff), during which hydraulic erosion develops.

Raindrops range in size from around 7.0 mm in diameter to fine mist sizes. The relation of raindrop size to intensity of rainfall has been established by Laws and Parsons (1943). Most rainfalls are considered to have a raindrop size distribution described by the normal or Gaussian probability distribution, based on raindrop volumes. Laws and Parsons define the median drop diameter, D_{50}, as the diameter where the total volume of water in the larger drops equals the total volume of water in the smaller drops and is used as a parameter to talk about rainfall drop sizes. Obviously, from the definition of D_{50} itself, it is clear that the number of raindrops larger than D_{50} is much less than the number of raindrops smaller than the median drop size. The choice of the term "median" is somewhat misleading. At any rate, the median drop size is related to rainfall intensity as follows:

$$D_{50} = 2.23\ I^{0.182} \qquad (1)$$

where I = rainfall intensity, in/hr

Raindrops reach a terminal velocity dependent on their diameter. These velocities are, for instance: 3.3 feet/second (1.01 m/sec) for a small drop 0.25 mm in diameter, 13.1 feet/second (3.99 m/sec) for a drop 1 mm in diameter, and 29.9 feet/second (9.13 m/sec) for a drop 5 mm in diameter. Actual impact velocity, however, is often greater than the terminal velocity due to wind effects imparting an added velocity component. Laws and Parsons (1943) published size distributions of raindrops as a percent of total volume for rainfall intensities ranging from 0.01 to 6.0 inches/hour (0.03 to 15.24 cm/hour). Gunn and Kinzer (1949) published the terminal velocities of water drops of different sizes in stagnant air.

From the relationship of kinetic energy to velocity (K.E. = $1/2\ mV^2$), the drop size distributions of rainfall determined by Laws and Parsons, and the terminal velocities of water drops of different sizes determined by Gunn and Kinzer, a least-squares linear regression analysis was performed by Wischmeier and Smith (1958) resulting in the following relationship between kinetic energy and rainfall intensity:

$$Y = 916 + 331 \log_{10} I \qquad (2)$$

where Y = kinetic energy in foot tons/acre-inch

I = rainfall intensity, in/hr

This equation and the maximum 30-minute storm intensity form the basis of the EI variable which is an integral part of the Universal Soil Loss Equation. Units shown must be used for computation; then the results may be converted to the metric system if desired.

Mutchler and Young presented some of their findings at the 1972 Sediment Yield Workshop, USDA Sedimentation Laboratory, Oxford, Mississippi. From plot studies and other observations, *raindrop impact* was found to be *most erosive* where a *very thin layer of water* is present (about 1/5 drop diameter) and is relatively *non-erosive* when the soil is covered with a *water depth of 3-drop diameters* or greater. Several statements are valid concerning soil detachment by rainfall:

(1) *raindrop splash* is the primary agent in soil detachment and transport from *inter-rill areas;*

(2) detachment is a result of *impact energy dissipation* on a saturated soil surface not protected by cover of plant material or sufficient water depth;

(3) the *transport* to rills is by *splash action alone* to a *limited extent* and *primarily* by very thin *surface flow* accelerated by raindrop impact splash velocities;

(4) runoff and sediment follow the same path down a slope from inter-rill area to rill area to larger water courses: true sheet flow rarely occurs;

(5) soil loss in a rill system is determined by the supply of detached soil particles by raindrop impact and then transport of these particles by the small channels—rill flow;

(6) at some point, the banks or channel bed of the watercourses dominate as a sediment source from a watershed.

Prediction of Erosion

The Universal Soil Loss Equation was derived from statistical analyses of soil loss and associated data obtained in 40 years of research by the Agricultural Research Service (ARS) and assembled at the ARS runoff and soil loss data center at Purdue University. The data include more than 250,000 runoff events at 48 research stations in 26 states, representing about 10,000 plot-years of erosion studies under natural rain. It was developed by Wischmeier and Smith (1965) as an *estimate of the average annual soil erosion from rainstorms for a given upland area,* expressed as the average annual *soil loss/unit area,* A (tons/acre):

$$A = (R)(K)(LS)(C)(P) \qquad (3)$$

where R = the rainfall factor

K = the soil erodibility factor

LS = the slope length gradient ratio

C = the cropping management factor or cover index factor

P = the erosion control practice factor

This equation represents the most comprehensive attempt at relating the major factors in soil erosion. It is used in the SWMM to predict the *average soil loss for a given storm or time period.* It is recognized that the Universal Soil Loss Equation was not developed for making predictions based on *specific* rainfall events. There are many random variables which tend to cancel out when computing annual time averages which would not cancel out when predicting individual storm yields: for example, the initial soil-moisture condition, or *antecedent moisture condition* (AMC), is a parameter which cannot be determined directly and used reliably. It should be understood by the SWMM user that Equation (3) enables land management planners to estimate *gross erosion rates* for a wide range of rainfall, soil, slope, crop, and management conditions.

Rainfall Factor, R

The rainfall factor, R, is equal to the *sum* of the rainfall erosion indexes for *all storms* during the period of prediction, $\sum EI$. For a single storm, R would simply equal EI for that storm. If we multiply Equation (2) by the total inches of rainfall received during each rainfall hyetograph time interval, and we sum over all the time intervals, then the *storm's total rainfall energy* is given by:

$$R = EI = \Sigma\ (9.16 + 3.31 \log X_i)D_i\ I \qquad (4)$$

where E = storm's rainfall energy (*hundreds* of foot-tons/acre)

$= \Sigma Y_i D_i = \Sigma(9.16 + 3.31 \log X_i)D_i$

i = rainfall hyetograph time intervals

X_i = rainfall intensity during time interval i, in/hr

D_i = inches of rainfall during time interval i

I = maximum average 30-minute intensity of rainfall

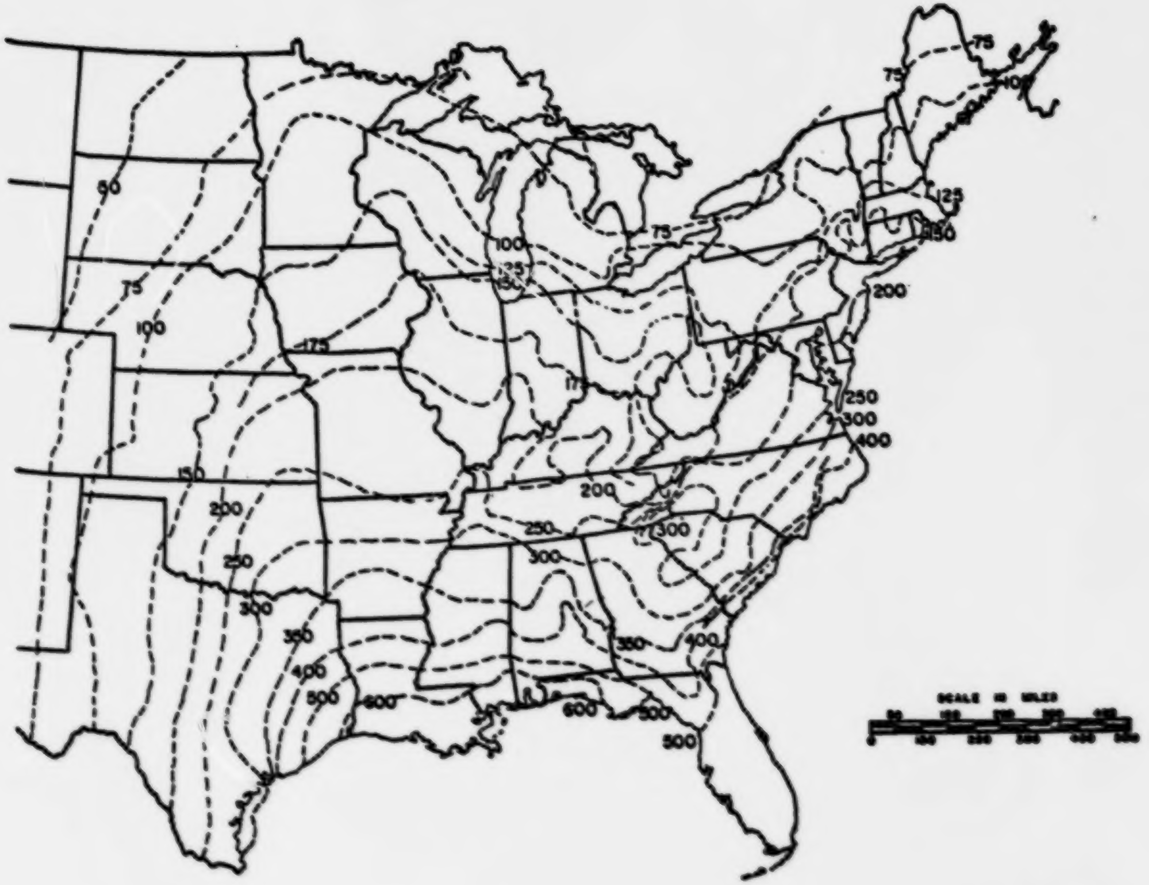
To avoid confusion over the interval rainfall intensity term and the maximum average 30-minute intensity of rainfall term in Equation (4), variable X was substituted for I in the expression for Y. Values for Y_i are given in Table 4 (Wischmeier and Smith, 1958) for different intensities, X_i. For predicting average annual erosion, values for the rainfall factor are shown in Figure 2 (Wischmeier and Smith, 1965) for locations east of the Rocky Mountains. It is very important to point out that, due to the effects of hurricanes and other tropical storms in the southeastern United States, the highest value of the erosion rainfall factor, R, that should be used is 350 (Ports, 1974, private communication). It is also worthy to note that the R factor *does not account for soil losses due to snowmelt and wind erosion.*

Where rainfall intensity records are not available for a single storm or a series of storms of interest, the erosion for these events can be calculated from the predicted annual erosion. Table 5 (Ports, 1973) gives values for a new variable, M, which is the fraction of annual erosion expected for a period of one to eleven months. Table 6 (Ports, 1973) lists values for estimating soil loss for storms of frequency less than one year and probable annual soil loss for one year in 2, 5, and 20 years. To obtain the expected soil loss these factors are simply multiplied by the predicted annual soil loss. Values from Table 5 and 6 apply for locations in Maryland, New Jersey, Delaware, eastern Virginia, and the eastern half of Pennsylvania. Similar tables can be developed for regions east of the Rocky Mountains from data in Agriculture Handbook 282 (Wischmeier and Smith, 1965).

Table 4. KINETIC ENERGY OF NATURAL RAINFALL*
(Hundreds of foot tons/acre inch)
(Wischmeier and Smith, 1958)

Intensity in/hr	Intensity, inches/hour									
	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.00	2.54	3.54	4.12	4.53	4.85	5.12	5.34	5.53	5.70
0.1	5.85	5.99	6.11	6.23	6.33	6.43	6.53	6.61	6.69	6.77
0.2	6.85	6.92	6.98	7.05	7.11	7.17	7.22	7.28	7.33	7.38
0.3	7.43	7.48	7.52	7.57	7.61	7.65	7.69	7.73	8.07	7.81
0.4	7.84	7.88	7.91	7.95	7.98	8.01	8.04	8.07	8.10	8.14
0.5	8.16	8.19	8.22	8.25	8.27	8.30	8.33	8.35	8.38	8.40
0.6	8.43	8.45	8.47	8.50	8.52	8.54	8.56	8.58	8.61	8.63
0.7	8.65	8.67	8.69	8.71	8.73	8.75	8.77	8.78	8.80	8.82
0.8	8.84	8.86	8.87	8.89	8.91	8.93	8.94	8.96	8.98	8.99
0.9	9.01	9.02	9.04	9.06	9.07	9.09	9.10	9.12	9.13	9.15
	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
1.0	9.16	9.30	9.42	9.54	9.64	9.74	9.84	9.92	10.00	10.08
2.0	10.16	10.23	10.29	10.36	10.42	10.48	10.53	10.59	10.64	10.69
3.0	10.74	10.79	10.83	10.88	10.92	10.96	11.00	11.04	11.08	11.12
4.0	11.15	11.19	11.22	11.26	11.29	11.32	11.35	11.38	11.41	11.44
5.0	11.47	11.50	11.53	11.56	11.58	11.61	11.64	11.66	11.69	11.71
6.0	11.74	11.76	11.78	11.81	11.83	11.85	11.87	11.89	11.92	11.94
7.0	11.96	11.98	12.00	12.02	12.04	12.06	12.08	12.09	12.11	12.13
8.0	12.15	12.17	12.18	12.20	12.22	12.24	12.25	12.27	12.29	12.30
9.0	12.32	12.33	12.35	12.37	12.38	12.40	12.41	12.43	12.44	12.46

* Example: Kinetic energy of rainfall of 0.57 in/hr = 8.35 hundreds of ft tons/acre in.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SCALE IN MILES

FIGURE 2.
Iso-Erodent Map (R Values for the Erosion Equation).
(Wischmeier and Smith, 1965)

Table 5. ADJUSTMENT FACTOR M FOR ESTIMATING MONTHLY AND PORTIONS OF ANNUAL SOIL LOSS FOR LOCATIONS IN MARYLAND, NEW JERSEY, DELAWARE, EASTERN VIRGINIA AND EASTERN PENNSYLVANIA

(Ports, 1973)

Starting month	Ending month											
	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Jan.	0.00	0.02	0.04	0.06	0.10	0.20	0.35	0.55	0.76	0.86	0.93	0.97
Feb.	0.98	0.00	0.02	0.04	0.08	0.18	0.33	0.53	0.74	0.84	0.91	0.95
Mar.	0.96	0.98	0.00	0.02	0.06	0.16	0.31	0.51	0.72	0.82	0.89	0.93
Apr.	0.94	0.96	0.98	0.00	0.04	0.14	0.29	0.49	0.70	0.80	0.87	0.91
May	0.90	0.92	0.94	0.96	0.00	0.10	0.25	0.45	0.66	0.76	0.83	0.87
June	0.80	0.82	0.84	0.86	0.90	0.00	0.15	0.35	0.56	0.66	0.73	0.77
July	0.65	0.67	0.69	0.71	0.75	0.85	0.00	0.20	0.41	0.51	0.58	0.62
Aug.	0.45	0.47	0.49	0.51	0.55	0.65	0.80	0.00	0.21	0.31	0.38	0.42
Sept.	0.24	0.26	0.28	0.30	0.34	0.44	0.59	0.79	0.00	0.10	0.17	0.21
Oct.	0.14	0.16	0.18	0.20	0.24	0.34	0.49	0.69	0.90	0.00	0.07	0.11
Nov.	0.07	0.09	0.11	0.13	0.17	0.27	0.42	0.62	0.83	0.93	0.00	0.04
Dec.	0.03	0.05	0.07	0.09	0.13	0.23	0.38	0.58	0.79	0.89	0.96	0.00

All dates are as of the first of each month. M = 1.0 for one full year.

Example: Given an average annual soil loss of 100 tons/acre/year. The estimated soil loss for July is 100 x 0.2 = 20 tons/acre. The estimated soil loss from May to September is 100 x 0.66 = 66 tons/acre.

Table 6. SOIL LOSS PROBABILITY AND SINGLE STORM SOIL LOSS FOR LOCATIONS IN MARYLAND, NEW JERSEY, DELAWARE, EASTERN VIRGINIA AND EASTERN PENNSYLVANIA (Ports, 1973)

Probability		Single storm	
One year in	Factor	Exceeded once in	Factor
2	0.90	1 year	0.2
5	1.25	2	0.3
20	1.70	5	0.4
		10	0.5
		20	0.7

Soil Factor, K

The soil factor is a measure of the potential erodibility of a soil and has units of tons/unit of erosion index, EI. The soil erodibility nomograph shown in Figure 3 (Wischmeier, Johnson and Cross, 1971) is used to find the value of the soil factor once five soil parameters have been estimated. These parameters are: percent silt plus very fine sand (0.05-0.10 mm), percent sand greater than 0.10 mm, organic matter content, structure, and permeability. To use the nomograph, enter on the left vertical scale with the appropriate percent silt plus very fine sand. Proceed horizontally to the correct percent sand curve, then move vertically to the correct organic matter curve. Moving horizontally to the right from this point, the first approximation of K is given on the vertical scale. For soils of fine granular structure and moderate permeability, this first approximation value corresponds to the final K value and the procedure is terminated. If the soil structure and permeability are different than this, it is necessary to continue the horizontal path to intercept the correct structure curve, proceed vertically downward to the correct permeability curve, and move left to the soil erodibility scale to find K. This procedure is illustrated by the dotted line on the nomograph. For a more complete discussion on this topic, see Wischmeier, Johnson and Cross (1971).

Table 7 (Maryland Water Resources Administration, 1973) lists soil factor values for soil types found in Maryland.

Slope Length-Gradient Ratio, LS

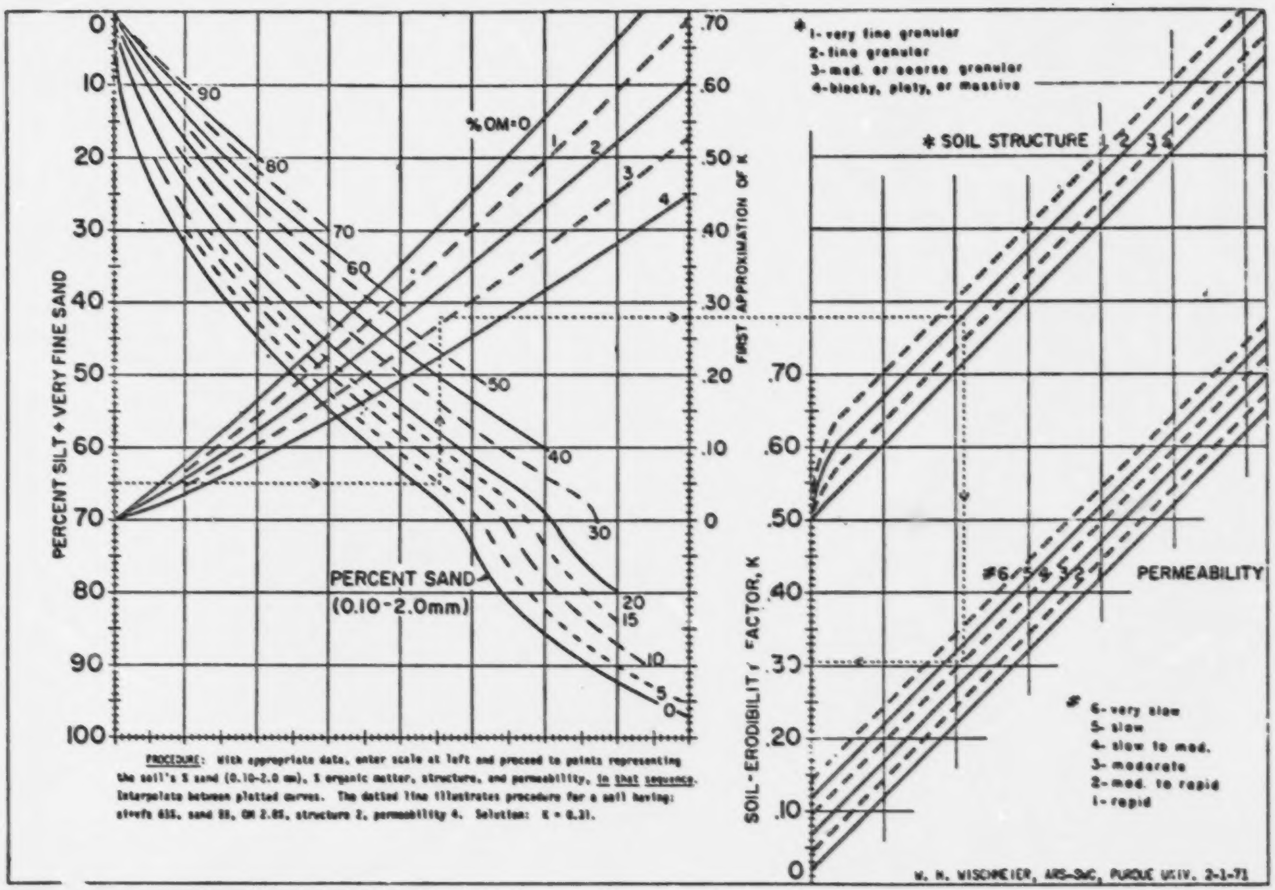
The slope length-gradient ratio is a function of runoff length and slope and is given by the following equation:

$$LS = L^{1/2}(0.0076 + 0.0053S + 0.00076S^2) \qquad (5)$$

where L = the length in feet from the point of origin of overland flow to the point where the slope decreases to the extent that deposition begins or to the point at which runoff enters a defined channel

S = the average percent slope over the given runoff length

Values for LS can be read from Figure 4 (Ports, 1973) knowing runoff length and slope. In using the average percent slope in calculating the LS factor, the predicted erosion will be different from the actual erosion when the slope is not uniform. Meyer and Kramer (1969)



PROCEDURE: With appropriate data, enter scale at left and proceed to points representing the soil's % sand (0.10-2.0 mm), % organic matter, structure, and permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: si+vfs 65%, sand 5%, OM 2.8%, structure 2, permeability 4. Solution: K = 0.31.

FIGURE 3. Soil Erodibility Nomograph.
(Wischmeier, Johnson and Cross, 1971)

Table 7. SOIL ERODIBILITY INDEX K VALUES FOR MARYLAND SOIL SERIES (Maryland Water Resources Administration, 1973)

Soil series	Horizon	Texture range	K Value
Adelphia	A	Sl,fsl,l	0.32
	B	L,scl,fsl	0.40
	C	Sl,ls	0.20
Athol	A	Sil	0.37
		Gsil,gl	0.32
	B	Sicl,cl	0.30
		G,cl	0.30
	C	Sicl,cl	0.30
		Gsl,gl	0.30
Aura	A	Sl,l	0.43
		Gl,gsl	0.30
		Ls	0.20
	B	Scl	0.40
		Gscl,gsl	0.30
	C	Scl,sl	0.40
		Gsl,gcl	0.30
		Ls	0.20
Bertie	A	Sil,l	0.37
	B	Sil,sicl,l	0.40
	C	Stratified	0.30
		Sl,l,ls	
		Gsl	0.20
Berks	A	Shsil,chsil	0.24
	B	Sh to vshsil	0.20
	C	Vshsil	0.20
		Shattered shale	0.20
Bermudian	A	Sil,l	0.43
		Fsl	0.40
	B	Stratified silt	0.50
		S	0.30
		G	0.20
Bibb	A	Sl to sicl	0.32
	B	Highly variable	0.20

Table 7 (continued). SOIL ERODIBILITY INDEX K VALUES FOR MARYLAND SOIL SERIES

Soil series	Horizon	Texture range	K Value
Birdsboro	A	Sil,l	0.28
	B	Sicl,cl	0.30
	C	Sl,s,g	0.20
		Sicl,l	0.30
Bucks	A	Sil	0.32
	B	Sicl,sil	0.40
	C	Shsil,vshsil	0.20
Chalfont	A	Sil,vstl	0.43
	B	Sil,sicl	0.60
	C	Shsil,shl	0.60
Chillum	A	Sil,sicl	0.32
		Gl	0.30
	B	Gscl,gl	0.30
		Gsl	0.20
Colemantown	A	L,sl	0.43
	B	Sc,scl	0.40
	C	Sl,cl,scl	0.40
Collington	A	Sl,fsl,l	0.28
		Ls	0.20
	B	Scl,cl,sl,l	0.40
	C	Sl,ls	0.20
Colts Neck	A	Sl	0.28
		Ls	0.20
	B	Scl,sl,l	0.40
	C	Sl	0.30
Croton	A	Sil	0.43
	B	Sil,sicl	0.50
	C	Shsil,shsicl	0.40
Donlonton	A	Fsl,ls,sil	0.43
	B	Sc,cl,sic	0.40
	C	Sc,sicl,cl,ls	0.30

Table 7 (continued). SOIL ERODIBILITY INDEX K VALUES FOR MARYLAND SOIL SERIES

Soil series	Horizon	Texture range	K Value
Duffield	A	Sil	0.32
	B	Sicl	0.30
	C	Sicl	0.40
		Shsil	0.30
Edgemont	A	Chl	0.24
	B	Chl,chscl	0.30
	C	Chl,shsl	0.20
Elkton	A	Sil,fsl,sl,l	0.43
	B	Sic,c	0.40
	C	Sic,sicl,scl	0.40
Evesboro	A	Ls,s	0.17
Fallsington	A	Sl,fsl,l	0.28
	B	Scl,sl	0.30
	C	S,ls,sl	0.20
Fort Mott	A	S,ls	0.20
	B	Sl	0.30
	C	S	0.20
Freneau	A	Sl,l	0.28
Galestown	A	Ls,s	0.17
Howell	A	Fsl,sil,l	0.43
	B	Cl,sicl	0.40
	C	C,sic,sicl	0.30
Keansburg	A	Sl,l	0.28
	B	Sl,l	0.30
Keyport	A	Sil,l,fsl	0.43
	B	C,sic,cl	0.40
	C	Sicl,sic	0.40
	Sandy substratum	Scl,sl	0.30
Klej	A	Ls,fs,lfs	0.17
	B	Ls,fs,lfs,sl	0.20

Table 7 (continued). SOIL ERODIBILITY INDEX K VALUES FOR MARYLAND SOIL SERIES

Soil series	Horizon	Texture range	K Value
Lakeland	A	Ls,lfs	0.17
Lansdale	A	L,sl	0.28
	B	Scl,sl	0.30
		L	0.40
	C	Chsil,gsl	0.30
		Chsl,gsl	0.20
Legore	A	Sil,sicl	0.24
		Gl	0.20
	B	Cl	0.30
		Gcl,gl,gsicl	0.20
	C	L,sil,sicl	0.30
		Gl,vgl,gcl	0.20
Lehigh	A	Sil	0.43
		Chsil	0.37
	B	Chsicl	0.40
	C	Chsicl,vchsil	0.30
Matapeake	A	Sil,fsl,l	0.32
	B	Sil,sicl	0.40
	C	S,ls,sl,l,gs	0.30
Matawan	A	Sl,ls,fsl	0.32
	B	Cl,scl,sc,sl	0.40
Mattapex	A	Sil,l,fsl	0.37
	B	Sicl,sil,cl	0.40
	C	Sl,ls,s,l,gs	0.20
Monmouth	A	Fsl,l,lfs	0.43
	B	Sc,scl	0.40
	C	Sl,scl,sc	0.30
Neshaminy	A	Sil	0.32
		Vstsil	0.28
	B	Sicl,cl,scl,sl	0.30
	C	Diabase bedrock	

Table 7 (continued). SOIL ERODIBILITY INDEX K VALUES FOR MARYLAND SOIL SERIES

Soil series	Horizon	Texture range	K Value
Norton	A	Sil,l	0.32
	B	Sicl	0.40
	C	Sil	0.40
		Vgl,shl	0.30
Othello	A	Sil,l,fsl,sicl	0.37
	B	Sicl,sil	0.40
	C	Sl,ls,scl	0.30
Penn	A	L	0.32
		Shsil	0.28
	B	Sil	0.40
		Shsil,sicl	0.30
Pocomoke	A	Sl,l,fsl,ls,lfs	0.28
	B	Ls,s	0.20
Raritan	A	Sil	0.43
	B	Cl,sicl	0.30
	C	Stratified silt,fsl	0.20
		C,sil,l,g	0.30
Readington	A	Sil	0.43
	B	Sil,sicl	0.40
	C	Sil	0.40
		Vshsil	0.30
Rowland	A	Sil,l	0.43
		Sicl	0.40
	B	Stratified silt and gravel	0.30
		Sil	0.40
Rutlege	A	Ls,lfs	0.17
	B	S,fs,ls,lfs	0.20
Sassafras	A	Fsl,l,sl,lfs	0.28
		Ls	0.20
		Gfsl,gsl	0.24
	B	Scl,sl,l	0.30
	C	Sl,ls,fsl,gsl,gls	0.20

Table 7 (continued). SOIL ERODIBILITY INDEX K VALUES FOR MARYLAND SOIL SERIES

Soil series	Horizon	Texture range	K Value
Shrewsbury	A	Sl,fsl,l	0.28
	B	Scl,sl	0.30
	C	S,ls,sl	0.20
Steinsburg	A	Sl	0.28
		Gsl,vgsl	0.24
	B	Gsl	0.20
	C	Sandstone	
Watchung	A	Sil	0.43
	B	C,cl,sicl	0.40
	C	Sil,sicl,l	0.40
Westphalia	A	Fsl,lfs	0.49
	B	Fsl,lfs,vfsl	0.40
	C	Fs,lfs,fsl	0.30
Woodstown	A	Sl,fsl,l	0.28
		Ls	0.20
	B	Scl,l,sl	0.40
	C	S,ls,sl,gsl,gls	0.20

USDA SOIL TEXTURE ABBREVIATIONS USED IN TABLE 7

C	Clay
Ch	Channery
Cl	Clay loam
Co	Coarse
Fs	Fine sand
Fsl	Fine sandy loam
G	Gravelly
Gcl	Gravelly clay loam
Gl	Gravelly loam
Gscl	Gravelly sandy clay loam
Gsl	Gravelly sandy loam
L	Loam
Lfs	Loamy fine sand
Ls	Loamy sand
S	Sand
Scl	Sandy clay loam
Sh	Shaly
Sic	Silty clay
Sicl	Silty clay loam
Sil	Silt loam
Sl	Sandy loam
St	Stony
Vfs	Very fine sand
Vfsl	Very fine sandy loam

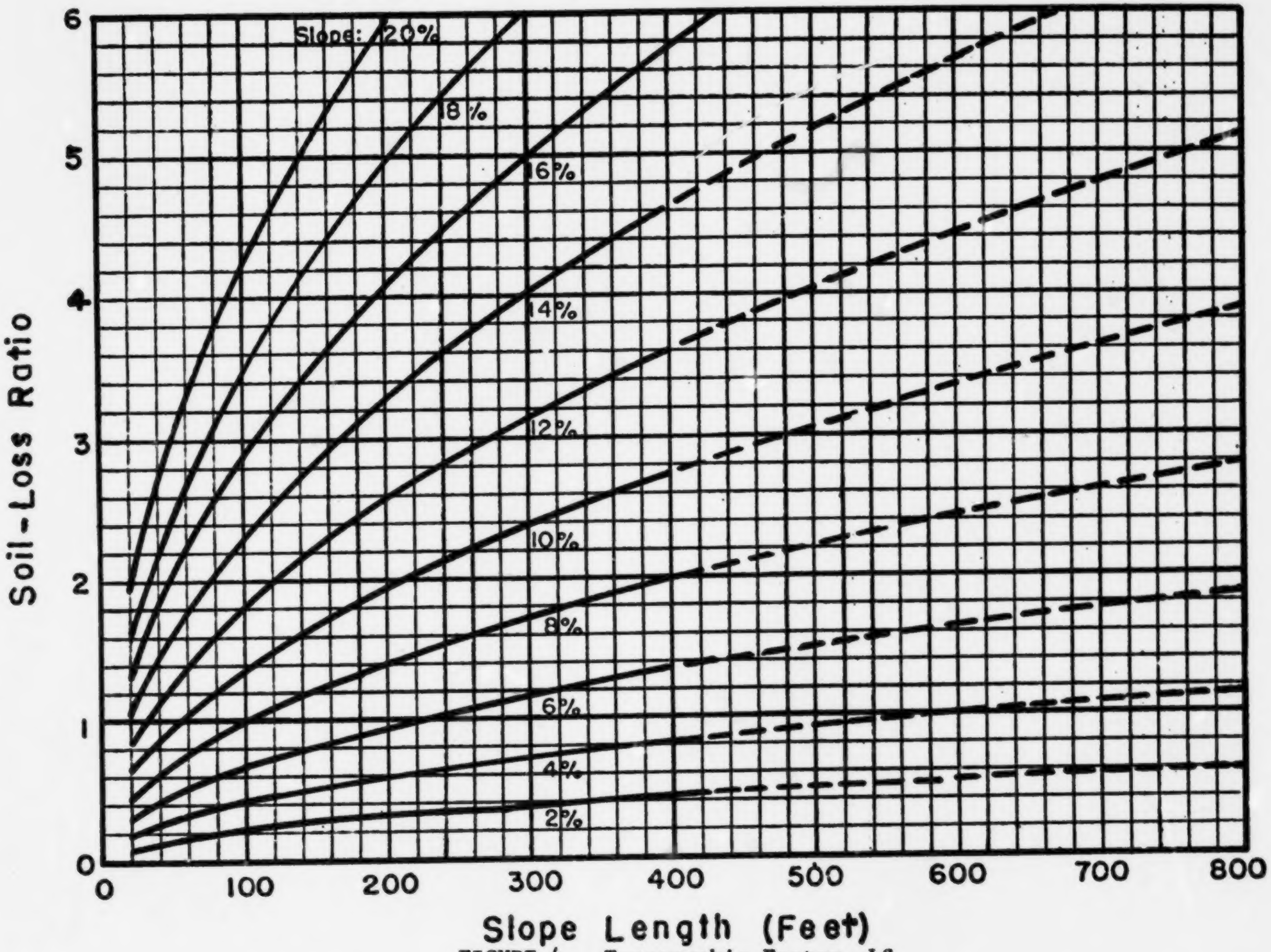


FIGURE 4. Topographic Factor, LS.
(Wischmeier and Smith, 1965)

show that when the actual slope is convex, the average slope prediction will underestimate the total erosion whereas for a concave slope the prediction equation will overestimate the actual erosion. If possible, to minimize these errors, large eroding sites should be broken up into areas of fairly uniform slope.

Cropping Management Factor, C

The cropping management factor is dependent upon the type of ground cover, the general management practice and the condition of the soil over the area of concern. The C factor is set equal to one for continuous fallow ground which is defined as land that has been tilled and kept free of vegetation and surface crusting. Values for the cropping management factor are given in Table 8 (Maryland Water Resources Administration, 1973).

Erosion Control Practice Factor, P

The control practice factor is similar to the C factor except that P accounts for the erosion-control effectiveness of superimposed practices such as contouring, terracing, compacting, sediment basins and control structures. Values for the control practice factor for construction sites are given in Table 9 (Ports, 1973). Agricultural land use P factor values can be found in Agriculture Handbook 282 (Wischmeier and Smith, 1965).

The C and P factors are the subject of much controversy among erosion and sedimentation experts of the US Department of Agriculture (USDA) and the Soil Conservation Service (SCS). These factors are *estimates* and many have *no theoretical or experimental justification*. It has been suggested that upper and lower limits be placed on these factors by local experts to increase flexibility of the Universal Soil Loss Equation for local conditions.

The P factors in the upper portion of Table 9 were designated as *estimates* when they were originally published. SCS scientists have found no *theoretical or experimental justification* for factors significantly greater than 1.0 (Soil Conservation Service, Hyattsville, Maryland, 1973). Surface conditions 4, 6, 7 and 9 ($P \leq 1.0$), Table 9, also are *estimates with no experimental justification.*

Sample Calculations

To demonstrate the use of the equation for sediment control planning on construction sites, a hypothetical one-acre residential development can be examined. The development site is located on the Conestoga silt loam soil, with an average slope of 10 percent and

Table 8. CROPPING MANAGEMENT FACTOR C
(Maryland Water Resources Administration, 1973)

Type of cover	C Value	Mulch	Rate of application (tons/acre)	C Value	Maximum allowable slope length
None (fallow)	1.00	Hay or straw	0.5	0.35	20 feet
			1.0	0.20	30
Temporary seedings:			1.5	0.10	40
First sixty days	0.40		2.0	0.05	50
After sixty days	0.05				
		Stone or gravel	15.0	0.80	15
Permanent seedings:			60.0	0.20	80
First sixty days	0.40		135.0	0.10	175
After sixty days	0.05		240.0	0.05	200
After one year	0.01				
		Chemical mulches			
Sod (laid immediately)	0.01	First ninety days	a	0.50	50
		After ninety days	a	1.00	50
		Woodchips	2.0	0.80	25
			4.0	0.30	50
			7.0	0.20	75
			12.0	0.10	100
			20.0	0.06	150
			25.0	0.05	200

[a] As recommended by manufacturer

Table 9. EROSION CONTROL PRACTICE FACTOR P FOR CONSTRUCTION SITES (Ports, 1973)

Surface condition with no cover	Factor P
1. Compact, smooth, scraped with bulldozer or scraper up and down hill	1.30
2. Same as above, except raked with bulldozer root raked up and down hill	1.20
3. Compact, smooth, scraped with bulldozer or scraper across the slope	1.20
4. Same as above, except raked with bulldozer root raked across slope	0.90
5. Loose as a disced plow layer	1.00
6. Rough irregular surface, equipment tracks in all directions	0.90
7. Loose with rough surface greater than 12" depth	0.80
8. Loose with smooth surface greater than 12" depth	0.90
Structures	
1. Small sediment basins:	
0.04 basin/acre	0.50
0.06 basin/acre	0.30
2. Downstream sediment basins:	
with chemical flocculants	0.10
without chemical flocculants	0.20
3. Erosion control structures:	
normal rate usage	0.50
high rate usage	0.40
4. Strip building	0.75

an overland flow length of 200 feet. From Table 7, the K value for the Conestoga series is found to be 0.43. The soil loss ratio is read from Figure 4 by locating the 10 percent curve and 200 foot slope length, LS = 1.93. For the moment, assume that no erosion control practice is instituted and that the soil is stripped of all vegetative cover during the construction process. For this situation, both C and P factors should be set equal to one. To calculate the expected annual erosion from this site, assume that the rainfall index value is 150 as read from Figure 2 for Lancaster County, Pennsylvania. Then,

$$A = (R)(K)(LS)(C)(P) \qquad (3)$$

$$\text{Annual Soil Loss} = (150)(.43)(1.93)(1)(1) = 124 \text{ tons/acre} = 279 \text{ MT/ha}$$

Assuming the period of construction before ground cover is applied to the site is four months, from April through July, factor M (Table 5) is used to calculate the soil loss.

$$\text{April-July Soil Loss} \quad (A)(M) = (124)(.29) = 36 \text{ tons} = 33 \text{ MT}$$

For a single storm whose rainfall hyetograph is shown in Figure 5 the rainfall factor, R, must be calculated according to Equation (4). The single storm R value is calculated with the use of Table 4 as shown below.

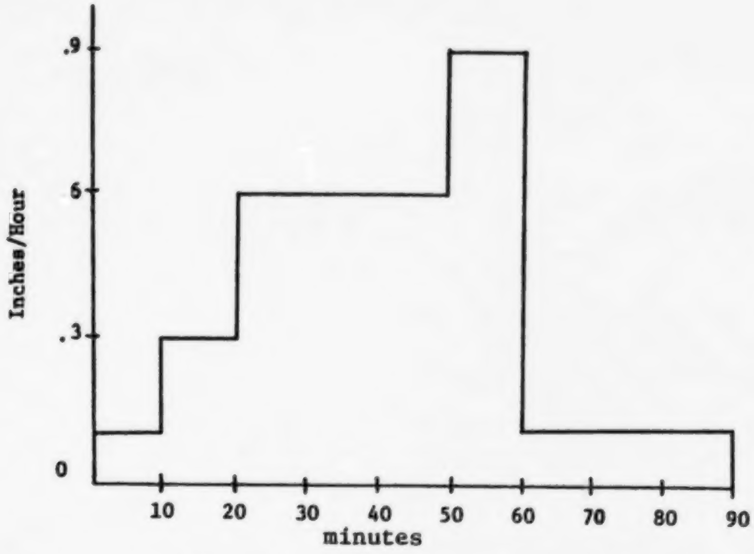


FIGURE 5. Rainfall Hyetograph

The expected soil loss from this one-acre site during this storm is:

$$\begin{array}{c}\text{Single Storm}\\ \text{Soil Loss}\end{array} = (3.25)(.43)(1.93) = 2.7 \text{ tons} = 2.45 \text{ MT}$$

According to Table 6, this hypothetical storm produces around one-tenth as much erosion as a one-year storm, as calculated below:

$$\begin{array}{c}\text{One-Year Storm}\\ \text{Soil Loss}\end{array} = (R)(K)(LS)(M) = (124)(.2) = 24.8 \text{ tons} = 22.5 \text{ MT}$$

By applying one ton of mulch over this one-acre construction site and placement of a downstream sediment basin, the erosion from this single storm event is reduced considerably.

$$\begin{array}{c}\text{Single Storm}\\ \text{Soil Loss}\\ \text{With Control}\end{array} = (2.7)(C)(P) = (2.7)(.15)(.2) = 0.08 \text{ tons} = 0.073 \text{ MT}$$

The percent effectiveness of these control measures is calculated from Equation (6) below:

$$\text{Percent Effectiveness} = (1-CP) \times 100 \qquad (6)$$

For this example, the effectiveness is 97%.

Summary of Limitations in the Use of the Universal Soil Loss Equation

The SWMM user should understand that the Universal Soil Loss Equation is an empirical equation based on statistical analyses of measured sediment yield from small plots. The sediment yield from these plots is, by definition, soil loss. The Universal Soil Loss Equation is being used successfully by the SCS to determine soil loss and to evaluate the effects of conservation measures on soil loss from farm fields. However, for larger areas, the Universal Soil Loss Equation gives only the *maximum potential sediment yield.* The actual yield will generally be less.

When the Universal Soil Loss Equation is used for computing *sediment yields* from *watersheds, all factors except the rainfall factor* must be modified (Williams and Berndt, 1972). Sediment yield may be defined as the amount of soil material eroded from a watershed that is transported and deposited in a stream either as *suspended sediment* or as *settled bed material,* or both. It is dependent on *gross erosion* in the watershed and *transport and deposition by runoff* into streams or reservoirs. The percentage of sediment delivered from the erosion source to any specified downslope location is affected by such factors as size and texture of erodible material, climate, land use, local environment, and physiographic position. The change (per unit area) of this downstream sediment movement, from the source to any given measuring point, is expressed by the *delivery ratio*, defined as: *the ratio of sediment yield at the measuring point to total material eroded from the watershed and drainage system upstream* from the measuring point. Thus, adequate determination of sediment yield from watersheds requires a *modified Universal Soil Loss Equation and an equation for predicting delivery ratios*.

The SWMM user is referred to Williams and Berndt (1972) for an in-depth discussion of this topic. It is important to note that successful use of this approach requires a multiple regression analysis to relate delivery ratios to watershed characteristics for local conditions. This requirement might result in considerable inconvenience to some SWMM users. Another alternative would be to use Flaxman's (1972) multiple regression equation. This equation relates sediment yield to four independent variables: one for the climate; one for the topography; and two for soil characteristics. However, the study area consisted of 11 western states (Arizona, California, Colorado, Idaho, Montana, Nevada, New Mexico, Oregon, Utah, Washington, and Wyoming), suggesting that use of this relationship outside of the study area would be risky. The study did demonstrate that a few parameters can delineate most of the variation in sediment yield in the western United States despite the great range in climate, topography, soils and geology, and land conditions. This is encouraging because it may also be true for other areas in the continental United States.

Wischmeier and Smith's relationship between kinetic energy and rainfall intensity, Equation (2), was developed for rainfall intensities ranging from 0.01 to 6.0 inches/hour (0.03 to 15.2 cm/hr). Modeling of storms having larger or smaller intensities constitutes an extrapolation of the least-squares linear regression relationship. Some of the other limitations of the Universal Soil Loss Equation pertain to the five major factors and have been discussed at length throughout the previous sections. We may summarize as follows:

1) a value of the rainfall factor R greater than 350 is not recommended;

2) the R factor does not account for soil losses due to snowmelt and wind erosion;

3) local soils experts should be consulted before selection of the C and P factors is made;

4) sediment yield from the watershed may be predicted only after modification of the Universal Soil Loss Equation and knowledge of the delivery ratios;

5) prediction is accurate only for storms with rainfall intensities ranging from 0.01 to 6.0 inches/ hour (0.03 to 15.2 cm/hr); and

6) some error is incurred when computing soil loss for an individual storm rather than using a time-averaged (annual) value.

The Universal Soil Loss Equation, as incorporated into the SWMM, can be applied successfully to *estimate* gross erosion rates from urban subcatchments or sub-basins for a wide range of rainfall, soil, slope, and cover conditions; provided, however, that the above limitations are recognized and observed. The predictive capabilities of the erosion modeling routines are enhanced by a better understanding of their range of applicability.

Incorporation of the Universal Soil Loss Equation into the Storm Water Management Model

An example has been shown of how the Universal Soil Loss Equation can be used to predict the total erosion from a given land area for a single rainfall event. Equations (3), (4) and (5) have been programmed and incorporated into the Storm Water Management Model so that suspended solids generated from eroding sites are added to the solids flushed from streets and other impervious surfaces. In the Model, the gross erosion for a specified subcatchment is calculated as follows:

1) the SWMM user specifies six input parameters which are constant for each subcatchment;

    a) the erosion area in acres;
    b) the flow distance in feet from the point of origin of overland flow to the point at which runoff enters the gutter or manhole, the runoff length;
    c) the soil factor, K;
    d) the cropping management factor, C;
    e) the control practice factor, P;
    f) the ground slope in ft/ft.

2) the SWMM user specifies the *time-step varying* rainfall hyetograph, inches/hour;

3) the SWMM user specifies the *storm varying* highest average 30-minute rainfall intensity, inches/hour;

4) the Model computes a rainfall index, at each time-step, using the appropriate rainfall intensity specified by the hyetograph;

5) the rainfall index, at each time-step, is then multiplied by the specified highest average 30-minute rainfall intensity to give the rainfall factor, R;

6) the rainfall factor, R, is subsequently multiplied, again at each time-step, by a constant representing the product of (LS)(K)(C)(P)(area) to give erosion in *tons;*

7) the tons of erosion are converted to a quality concentration, suspended solids in mg/l, from knowledge of the flow magnitude at each time-step; and

8) the suspended solids concentration due to erosion is added to the suspended solids concentration due to other sources, and both the total suspended solids concentration and the suspended solids concentration without erosion are printed for each time interval as shown in Table 10, columns 4 and 10, respectively.

C. VEGETATION

The literature contains many studies on the quantity of stormwater runoff. Several studies have been done that amply demonstrate the poor quality of such urban runoff and stormwater overflows. However, little mention has been made of the sources of this water pollution. In a Chicago study, it was demonstrated that vegetation could account for a large percentage of street solids on an annual basis (Heaney and Sullivan, 1971). Considering vegetation or leaf litter as a possible source of water quality impairment, the following section will be devoted to characterizing this material by quantifying amounts of litter produced, its chemical and nutrient content, and its potential effects on water quality.

There is a wealth of data on leaf litter production. However, most of this information is taken from forestry data and represents a closed-canopy environment. Most urban areas are far from a closed-

Table 10. EROSION PRINTOUT

```
        LANCASTER  PENNSYLVANIA  STEVENS  AVE  DISTRICT ****  WRE VERSION  ****
        STORM OF NOVEMBER 29, 1971 DURATION 6.5 HRS. STUDY 88 (STORM 88)

     SUMMARY OF QUANTITY AND QUALITY RESULTS AT LOCATION   31

     FLOW IN CFS AND QUALITY IN (MG/L)

      TIME        FLOW       BOD      SUS-S     COLIF      SET-S     NIT      PO4     GREASE   SUS-S WITHOUT EROSION
  11 40.00        3.32     62.47   6304.61  55675.59      0.02     0.00     0.04      0.47          4.75
  12  5.00        0.23     62.58   1717.01  54989.88      0.02     0.00     0.04      0.46          4.59
  12 30.00        0.53     62.28    270.69  52135.68      0.02     0.00     0.04      0.44          4.35
  12 55.00        0.56     61.44     90.94  47450.90      0.02     0.00     0.04      0.40          4.09
  13 20.00        0.58     61.23    275.71  44647.11      0.02     0.00     0.03      0.38          3.81
  13 45.00        0.60     60.73    225.87  41241.18      0.02     0.00     0.03      0.35          3.51
  14 10.00        0.32     60.11    152.32  37830.20      0.01     0.00     0.03      0.32          3.20
  14 35.00        1.17     59.22    295.65  32862.13      0.01     0.00     0.02      0.27          2.77
  15  0.0         1.33     58.25    131.60  27856.00      0.01     0.00     0.02      0.23          2.38
  15 25.00        1.53     57.21    224.08  21291.28      0.01     0.00     0.02      0.20          1.90
  15 50.00        1.77     55.80     88.36  16298.07      0.01     0.00     0.01      0.15          1.54
  16 15.00        2.71     50.18    259.34  14163.15      0.01     0.00     0.01      0.12          1.21
  16 40.00        2.80     52.42    230.88  10380.81      0.00     0.00     0.01      0.09          0.85
  17  5.00        2.74     53.96    204.08   7543.88      0.00     0.00     0.01      0.06          0.65
  17 30.00        1.57     49.40     93.65   5619.75      0.00     0.00     0.00      0.05          0.48
  17 55.00        0.75     48.44      0.91   4830.19      0.00     0.00     0.00      0.04          0.41
  18 20.00        0.29     48.39      0.39   4601.91      0.00     0.00     0.00      0.04          0.39
  18 45.00        0.11     48.30      0.39   3513.94      0.00     0.00     0.00      0.04          0.38
  19 10.00        0.38     48.25      0.39   4404.45      0.00     0.00     0.00      0.04          0.38
  19 35.00        0.32     48.22      0.38   4443.10      0.00     0.00     0.00      0.04          0.38
  20  0.0         0.32     48.20      0.38   4426.39      0.00     0.00     0.00      0.04          0.38
  20 25.00        0.32     48.19      0.38   4418.72      0.00     0.00     0.00      0.04          0.38
  20 50.00        0.31     48.19      0.38   4406.27      0.00     0.00     0.00      0.04          0.38
  21 15.00        0.31     48.17      0.38   4399.91      0.00     0.00     0.00      0.04          0.38
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canopy forest situation, but with the use of some basic tree density data, a correlation could be obtained to estimate the total leaf litter production in a typical urban community.

Tree Litter Production

In their review of the literature on litter production by the world's forests, Gorham and Bray (1964) state that "the study of the quantitative aspects of litter fall remains an important part of forest ecology, dealing with a major pathway for both energy and nutrient transfer in this type of ecosystem." In the urban environment, this energy and nutrient transfer might be disrupted (resulting in removal of nutrient energy) when this organic material or fuel for the nutrient cycle is not allowed to reach the upper soil horizons of such an ecosystem.

In a work by Chandler (1941), he discusses the studies of Ebermayer (1876) and others concerning the amount of dry matter in the annual litter fall of mixed hardwood forests, composed principally of European beech with an admixture of oak and birch. The values of dry matter ranged from 2,300 pounds to 5,000 pounds/acre (2577 to 5602 kg/ha) with an average of approximately 3,123 pounds (3499 kg). Such figures appear to be nominal for this climate. Gorham and Bray (1964) indicate from their review of the literature that as much as 13,500 pounds of litter/acre-year (15,125 kg/ha/yr) might be deposited in equatorial forests of the Congo. Annual litter production can be based on broad climatic zones: Equatorial, Warm Temperate, Cool Temperate, and Arctic-Alpine. The Warm Temperate group ranges between 30° and 40° both north and south of the Equator and includes the southern part of the United States, *e.g.*, Florida, the Carolinas, Tennessee, and California. The Cool temperate areas in North America range from Missouri and the mountains of California to Minnesota and Quebec, or about latitude 37-47°N. Within these broad climatic regions, Gorham and Bray (1964) have compiled annual leaf litter production data. For some of the areas in North America, the data are represented in Table 11. In general, it is recognized that leaf material contributes 60 to 70 percent of the origin of forest litter as shown in Table 12.

The Angiosperms are the dominant plant life of the geological era in which we live. This grouping contains the broad-leafed hardwoods and other flowering plants, while Gymnosperms are mainly pines or cone-bearing trees having needles. The Gymnosperms, together with the Angiosperms, are the seed plants. Many other forms of flora are also included in these divisions. Some of the tree species encountered in this study and in Table 11 are listed in Table 13 with both their common and scientific names.

Table 11. ESTIMATED AMOUNT OF TREE LITTER IN VARIOUS LOCATIONS IN THE UNITED STATES (Daubenmire, 1953; Gorham and Bray, 1964; Heyward and Barnette, 1936)

Location	Latitude	Longitude	Plant community	Litter fall (lb/acre/yr) Leaves	Other	Total
California	39N	123W	*Pinus ponderosa*	1,873		
Tennessee	36N	84W	mixed *Quercus* spp.			2,587
Tennessee	36N	84W	*Pinus virginiana*	3,390	624	4,014
			Quercus spp.	3,122	892	4,014
N. Florida	30N	83W	*Pinus palustris* 360 trees/acre	2,408		
			Pinus palustris 470 trees/acre	3,032		
			Pinus palustris 788 trees/acre	2,408		
			Pinus palustris 972 trees/acre	2,943		
S. Carolina	35N	82W	*Pinus echinata* and mixed angiosperms	3,390	1,427	4,817
Montreal	47N	74W	*Betula populifolia*	1,516		
Minnesota	47N	95W	*Pinus banksiana*	1,873		
			Pinus banksiana and *Pinus resirosa*	1,784		
Missouri	39N	92W	*Pinus echinata*			3,390
New York	43N	77W	*Tilia Americana* and mixed angiosperms	2,765		
New York	43N	76W	*Pinus strobus*	2,765		
Wisconsin	43N	89W	*Quercus alba* and *Quercus velutina* 170 trees/acre	4,103	1,338	5,441
Connecticut	42N	73W	*Pinus strobus*			3,568
			Acer sacchorum *Quercus rubra* and mixed angiosperms			1,873

Table 12. SOURCES OF FOREST LITTER

Source	% of total litter
Leaves	60-70
Branches	12-15
Bark	1-14
Fruit	1-17

Table 13. COMMON AND SCIENTIFIC NAMES OF TREES

Common name	Scientific name
American Elm	*Ulmus americana* L.
Basswood	*Tilia americana* L.
Red Maple	*Acer rubrum* L.
Sugar Maple	*Acer saccharum* Marsh
White Oak	*Quercus alba* L.
Red Oak	*Quercus borealis*
White Pine	*Pinus strobus* L.
Red Pine	*Pinus resinosa* A.T.
Ponderosa Pine	*Pinus ponderosa* Dougl.
White Ash	*Faxinum americana* L.
Black Birch	*Betula lenta* L.

Because of their evergreen nature, Gymnosperms might be expected to be more productive than deciduous Angiosperm trees, although this factor may be countered to some extent by the tendency for Angiosperms to occupy more fertile sites (Gorham and Bray, 1964). As far as litterfall is concerned, Table 14 indicates that when a wide range of sites are considered, Gymnosperms yield about one-sixth more total litter annually than Angiosperms, the difference amounting to 446 pounds/acre (500 kg/ha). The difference for leaf litter alone is approximately 8 percent or 178 pounds/acre.

This trend toward greater litter production from evergreen trees is observed in other studies. Chandler calculates the amount of dry matter deposited annually by hardwood forest tree leaf litter in closed, second-growth stands in central New York ranged from 2,425 to 3,020 pounds/acre (2,717 to 3,384 kg/ha) (Chandler, 1941). In a similar study, he determined the amount of dry needles from northeastern conifers was 2,009 to 3,367 pounds/acre (2,251 to 3,772 kg/ha) (Chandler, 1944). These figures he feels are typical of healthy vigorous stands. Another interesting point to be made from Chandler's work is that even though conifers shed needles during the entire year the greatest drop of needles occurs along with that of deciduous trees in autumn and winter.

From the combined data by Gorham and Bray in their exhaustive work they have been able to determine annual litter production in the four major climatic zones and have correlated these production figures to latitude. These data are represented in Table 15 and Figure 6.

We are interested in relating this type of data to an environment where the trees do not represent a closed-canopy forest situation. If tree density is examined, we find little effect by density differences within a closed-canopy forest (Gorham and Bray, 1964). This can be seen in the data by Heyward and Barnetts (1936). The results of this north Florida study were presented in Table 11. Several studies have shown a distinct correlation between annual fall of leaf litter and stand basal area in both Gymnosperm and Angiosperm stands (Gorham and Bray, 1964). Basal area is defined as the cross-sectional area of the trunk of a tree at breast height. This basal area for a tree species is roughly proportional to its crown size.

Studies of this type have shown that litter fall averaged about 70 to 75 kg/m^2 over a basal area range from 8 to 40 m^2/ha (Gorham and Bray, 1964). This represents approximately 14.3 to 15.3 pounds/feet^2 over a basal area range from 34.9 to 174.4 feet^2/acre.

Also in their literature review, Gorham and Bray have shown that in a forest system, thinning results in a decrease of litter production

Table 14. A COMPARISON OF LITTER PRODUCTION BY EVERGREEN AND DECIDUOUS TREES IN THE NORTHERN HEMISPHERE
(Gorham and Bray, 1964)

	No. regions averaged	Evergreen gymnosperms	Deciduous angiosperms
		(lbs/acre-yr)	
Total litter	8	3,300	2,854
Leaf litter	9	2,319	2,141
by difference		981	713
Other			
observed	4	624	624

Table 15. ANNUAL LITTER PRODUCTION IN FOUR MAJOR CLIMATIC ZONES
(Gorham and Bray, 1964)

	Leaves		Other		Total	
	No. regions averaged	lbs/acre	No. regions averaged	lbs/acre	No. regions averaged	lbs/acre
Arctic-Alpine	1	624	1	357	3	892
Cool Temperate	15	2,230	10	803	22	3,122
Warm Temperate	8	3,211	5	1,695	7	4,906
Equatorial	2	6,066	1	3,122	4	9,723

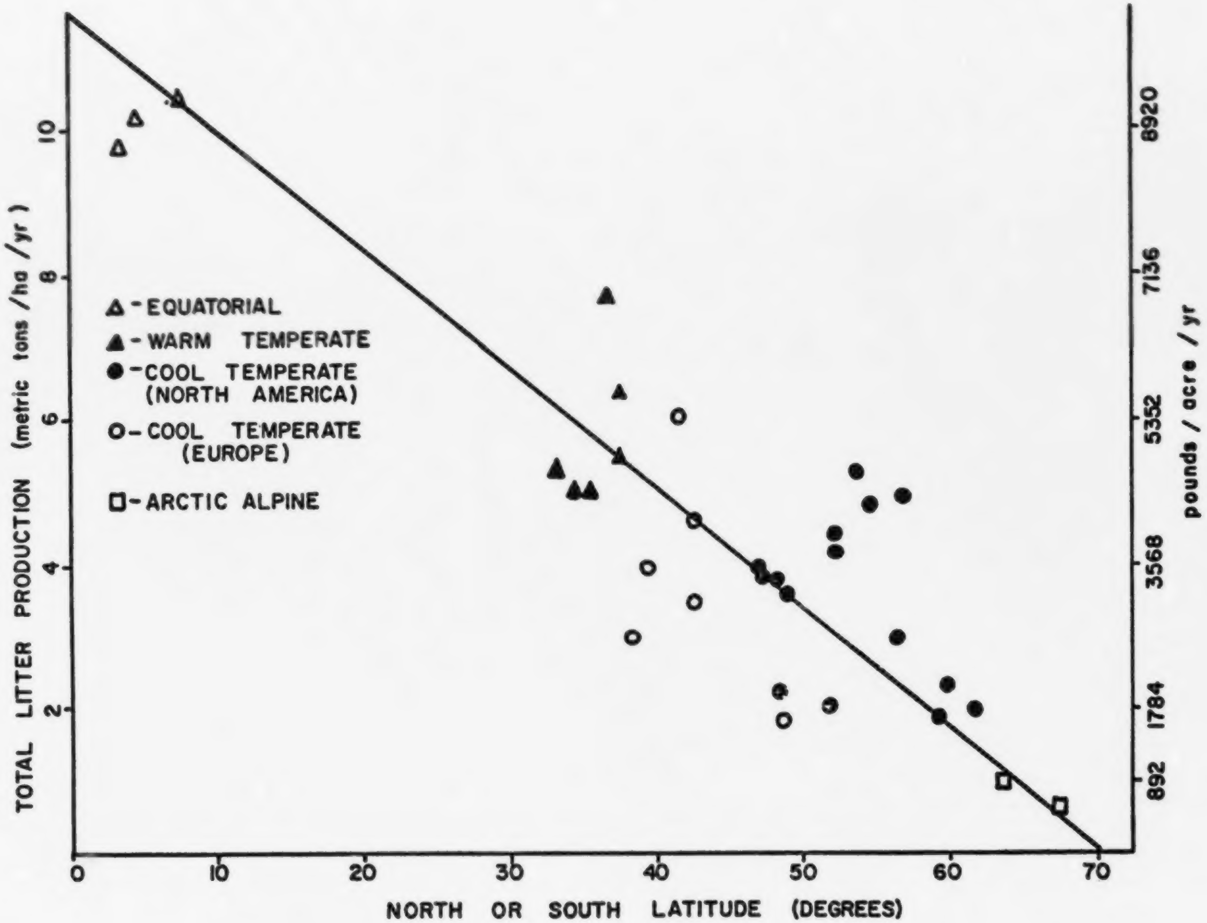

FIGURE 6.
Annual Production of Total Tree Litter in Relation to Latitude.
(Gorham and Bray, 1964)

that is roughly proportional to the degree of thinning. Another effect worth mentioning is the effect of litter removal from a forested area which causes extreme reduction of forest growth (Mayer-Krapoll, 1956). The loss of nitrogen in the litter is believed to be significant in lessening forest productivity. In light of this fact, litter that is removed from an area where the nutrients can be utilized represents a substantial amount of nutrients introduced into another system. In an urban community, it would appear that litter not retained on the soil and finding its way to some impervious surface could contribute to the nutrient load of runoff. This area of potential effects from litter will be discussed later in this section. It was also pointed out by Gorham and Bray that the age of the stand had little effect on total annual leaf-litter production. Other studies have confirmed their review.

Work by Carlisle *et al.* (1969) indicates similar results to those summarized by Gorham and Bray. In this study, the wooded area was stocked with uneven-aged (about 40 to 120 years) trees with a 90 to 95 percent closed-canopy and 64 trees/acre (158 trees/ha). The tree species were mainly oak and birch. The mean annual rainfall of this area was 67.5 inches (171.5 cm). Over a four-year study period, an average of 9.56×10^6 leaves/acre/year (2.36×10^7 leaves/ha/yr) were produced with a mean dry weight (seasonal maximum) of 3,407 pounds/acre (3,817 kg/ha). The dry weight of the total litter fall (leaves, twigs, flowers, acorns, etc.) was 3,441 pounds/acre (3,855 kg/ha). The autumn leaf fall averaged 1,967 pounds/acre (2,204 kg/ha) or 57.76 percent of the total annual leaf fall. This represents approximately 53 pounds of leaves per tree annually (24 kg of leaves per tree annually). Recognizing that there is no significant correlation between litter fall and tree density, this figure, however, represents the higher end of a broad spectrum of values that could be obtained from similar calculations.

If we now can consider that we have some feel for the amount of litter fall produced annually by trees, let us seek to determine the nutrient content of such matter.

Nutrient Content

The literature contains a liberal quantity of data on the nutrient content of litter. This section reviews only those findings pertinent to our discussion and that best exemplify the trends.

Considerable interest has been directed to the study of chemical composition of tree leaves. It has been recognized for some time that the composition of leaves varies with species, although it may be modified to some extent by variation in site. Daubenmire (1953) concluded that the chemical composition of tree leaves is primarily a characteristic of the species and substantiates the general conclusion reached by earlier workers.

Forest liter is not wholly organic, but always contains some mineral matter. Gorham and Bray (1964) have summarized some of this mineral content data. They found that the Angiosperms contained more mineral material than the Gymnosperms. The majority of the Gymnosperms had from 2 to 5 percent ash content while the Angiosperms ranged from 4 to 14 percent ash. Mean ash content as a percentage of dry matter of 19 Gymnosperm species was 3.7 percent and of 56 species of Angiosperms, 8.4 percent. They also noted that the ash content may vary slightly with region, owing mainly to soil differences. A detailed analysis of the major elements comprising mineral material in litter are numerous, and have been reviewed extensively by Lutz and Chandler (1946). However, the scope of this paper limits this data to some broad ranges. Daubenmire (1953) found that on his analyses for N, P, K, and Ca, values ranged from 0.3 to 0.7 percent for N, 0.04 to 0.24 percent for P, 0.13 to 1.19 for K, and 0.19 to 2.19 percent for Ca. These percentages represent total content of oven-dry weight of freshly fallen foliage. In an earlier study by Coile (1936), he determined that leaves of white oak contain 1.25 percent N. Those of red cedar, black oak, dogwood, shortleaf pine, loblolly pine, yellow poplar, and red gum contain progressively less to a minimum of 0.5 percent for red maple. Tables 16 and 17 give some average values for the nutrient content of newly fallen Gymnosperm and Angiosperm foliage. Consistent with these nutrient values were those of Carlisle *et al.* (1966) with N and P values of 18.9 pounds/acre/year and 1.1 pounds/acre/year (21.2 and 1.23 kg/ha/yr) respectively.

Leaf age was considered by Morrison in a recent study. He found that with increasing leaf age, P, K, and Mg concentrations decreased whereas Ca, Fe, and Mn increased. N showed no significant trend with leaf age. Other recent work was that of Kawahara (1972). He reported litter fall and nutrients throughout a year in six forests. The amount of total litter fall was 2,033 to 6,282 pounds/acre/year (2,277 to 7,038 kg/ha/yr). Leaf litter accounted for 54 to 78 percent in coniferous forests and as much as 83 percent in deciduous forests. The total litter-fall/year contained 13.6 to 64.7 pounds/acre N (15.2 to 72.5 kg/ha), 0.6 to 4.5 pounds/acre P (0.7 to 5.0 kg/ha), 3.2 to 14.6 pounds/acre K (3.6 to 16.4 kg/ha), 19.9 to 64.8 pounds/acre Ca (22.3 to 72.6 kg/ha), and 2.4 to 12.3 pounds/acre Mg (2.7 to 13.8 kg/ha). N, P, and K were more abundant in broad-leaved forests than in coniferous forests. He also computed the ratios of nutrients return with total litter fall to nutrient uptake as 33 to 52 percent for N, 36 to 69 percent for P, 23 to 43 percent for K, 62 to 85 percent for Ca, and 52 to 98 percent for Mg.

In their 1969 study, Moir and Grier found pine forest floor humus weights ranged from 11.5 to 18.5 tons/acre/year. This humus and organic accumulation of the forest floor was determined to be almost entirely derived from lodgepole pines. The reported nutrients from their study

Table 16. THE CONCENTRATION OF NUTRIENTS IN NEWLY FALLEN GYMNOSPERM AND ANGIOSPERM TREE LEAF LITTER
(% Dry Weight)
(Carlisle and White, 1966)

	Author	No. Species	Location	N	P	K	Ca	Mg	Ash
Gymnosperms	Lutz and Chandler (1946)	9	U.S.A.	0.58 to 1.25	0.04 to 0.10	0.12 to 0.39	0.55 to 2.16	0.14 to 0.23	3.01 to 4.33
Angiosperms	Lutz and Chandler (1946)	14	U.S.A.	0.51 to 1.01	0.09 to 0.28	0.40 to 1.18	0.99 to 3.64	0.22 to 0.77	5.71 to 15.16
Angiosperms	Duvigneaud & Denaeyer-deSmet (1964)	25	Europe	0.95 to 4.60	0.11 to 0.82	0.22 to 4.30	1.00 to 5.50	0.15 to 0.80	6.20 to 27.30

Table 17. AVERAGE QUANTITIES OF NUTRIENTS FALLING IN THE LITTER OF DIFFERENT TREES
(lbs/acre/yr)
(Carlisle and White, 1966)

Trees leaves only	Author	N	P	K	Ca	Mg
Angiosperms	Lutz and Chandler (1946)	13.2	2.5	10.7	52.2	7.3
Gymnosperms	Lutz and Chandler (1946)	18.8	1.4	5.2	21.1	3.6

are as follows: total N ranged from 290 to 480 pounds/acre (325 to 538 kg/ha), total P ranged from 29 to 44 pounds/acre (33 to 49 kg/ha), total K ranged from 62 to 151 pounds/acre (69 to 169 kg/ha), and total Ca ranged from 89 to 222 pounds/acre (100 to 249 kg/ha). Nutrient quantities, like these and others, indicate substantial amounts that could potentially contribute to the load of a receiving water. This material could directly fall into bodies of water, it could be transported by wind or runoff, or it might have such nutrients leached from it and then be transported in solution to a receiving body.

Normally leaves fall from trees, becoming part of the forest floor or humus, and then begin to decay and decompose. The rates of decomposition vary among species and with varying chemical composition, temperature, moisture, and various other physical conditions. The results of such findings can be found in works by Melin (1930), Kucera (1959), Hayes (1965) and others. Of major interest to this report is the decomposition and nutrient release of this leaf litter as it affects water quality.

Effects on Water Quality

Previous studies have demonstrated impaired water quality by the addition of natural organic matter to small streams or pooled streams. Of interest to this subject are studies by Slack (1964) and Slack and Feltz (1968). From his study of the Cacapon River, Slack attributes changes in water color and changes in water composition to the accumulation of litter at low flow. He reports an increase in the concentrations of some solutes with a decrease in water quality. Dissolved oxygen decreased with the increase in water color. Chase and Ferullo (1958) showed their leaves deplete dissolved oxygen under aerobic conditions in the laboratory. After 386 days, maple leaves had consumed a weight of oxygen equivalent to 75 percent of their initial dry weight. Slack indicates that an increase in stream flow tends to reduce color and improve water quality.

The results reported by Slack and Feltz in their 1968 study indicate similar findings. In addition to their field studies, laboratory tanks were used to evaluate the effects of various species of leaves. They reported that the greatest effects of leaf litter on water quality occur during chemical and biological dissolution of the submerged leaves. Previous study has indicated that leaf litter decomposes more rapidly in the presence of moisture (Heyward, 1936). Studies have shown that the solute load of streams can be altered rapidly by leaf fall. They also indicate that the degree of water quality degradation is directly related to the weight of litter/unit volume of water and to species composition of litter. In a 1933 study by Haupt (1933), he reported his laboratory studies on the underwater decomposition of leaves of various trees. He concludes his report by advising that possible reservoirs be protected from leaf contamination. He states that pines are best for forests around reservoirs, and that oaks are especially objectionable.

In the urban environment, these types of studies lend themselves to considerations of stormwater systems and runoff water quality. If such organic material enters a storm sewer or catchbasin where shallow stagnant pools exist, serious water quality impairment may result. From our previous discussion of litter, it should be recognizable that tree litter may constitute significant fractions of organic material in storm runoff in an urban area. We have also considered the tremendous quantities of nutrients available from such litter that can be readily dissolved. Nutrients leached from leaves and vegetation by precipitation are either returned to the soil or removed in runoff water. White (1973) reports the nutrient enrichment of runoff may be enhanced during the fall and winter by the rupturing of plant cells by freezing. In his study, he indicates that freezing increased nutrient release if the vegetation were growing when it was frozen. Newly fallen litter not yet decomposed might be affected in a similar manner. These findings are of additional concern in northern climates with snow melting and thawing contributing to runoff.

In another study, leaf leachates, simulating canopy drip during rainfall, were collected from southern red oak and longleaf pine (Malcolm and McCracken, 1968). From the representative canopy drip sampled, it was estimated that approximately 18 pounds/acre/year (20 kg/ha/yr) of organic matter could be contributed to the soil from this source. Considering these findings, large quantities of nutrients could be added to the organic load of urban runoff if it is permitted to fall upon an impervious surface. With the amount of nutrients that trees potentially return to the soil in the form of either leachate or litter capable of being leached by precipitation, it would seem unwise to create impervious areas immediately beneath trees or vegetation.

Grass Litter

In addition to the leaf litter that might be contributed by tree litter fall, grass litter is also a potential source of organic material. When we refer to grass litter, we are referring to grass clippings that result from the production and harvesting of lawns or turf grasses. As a lawn produces foliage, it is clipped or mowed to maintain its attractive appearance. Well-manicured herbage is common in most urban and residential areas and is, again, a source of civic pride and pleasure.

However, the disposal of the clippings or discharge of such litter in such a manner as to find its way into runoff of an area could be a source of pollution. As in the case of tree litter, grass litter is organic material, composed of various nutrients, and readily available for decomposition. It has been recognized that grasses act as filters and entrap solid materials of small sizes that fall among them. If minimal grass clippings result from regular mowings, this litter, if allowed to remain on the upper zone of the lawn, will soon find its way down among the blades or leaves and gradually decompose. Among turf specialists, it has been observed that as the clippings decompose, their nutrients are readily incorporated into the growing herbage. It is also felt that a minimal amount of lateral nutrient movement occurs in the natural filter. When this litter is not allowed to remain and return its nutrients to the root structure but is removed and not managed properly, nutrient enrichment of urban runoff is possible.

Once again, we consider the sweeping or dumping of this litter onto an impervious surface where stormwater runoff can expedite decomposition and leach nutrients into the stormwater system.

Such litter can be the source of organic material and nutrients as can be seen by the following information. In a Florida study (Institute of Food and Agricultural Sciences, University of Florida, April 1972), summarized data for several years showed that numerous ryegrass varieties averaged from 3,676 to 5,612 pounds of dry matter/acre/year (4,118 to 6,288 kg/ha/yr). In the same report, white clovers (primarily a forage crop but found in some lawns) averaged between 3,805 to 5,108 pounds of dry matter/acre/year (4,263 to 5,723 kg/ha/yr) over a 13-year study. In a study by Ruelke and Prine (1971), yields of various forage grasses were determined. Several of these are also commonly used as turf grasses for lawns. They found that Pensacola bahia grass could yield an annual average of 8,126 pounds/acre (9,160 kg/ha) during their five-year study. Coastal bermudagrass was found to produce an annual yield of 9,135 pounds of dry matter/acre (10,235 kg/ha) in that same study. It was observed that yields of dry matter were proportional to levels of fertilization. The response to amounts of nitrogen, phosphorous, potassium, calcium, and magnesium removed in the harvested forage was significantly greater as fertility levels are increased. The five-year mean first cutting dry matter yields of Coastal bermudagrass over all fertility levels was 2,542 pounds/acre (2,848 kg/ha). Such data probably represent the upper limits for yields of fertilized grasses common to Florida as forage crops and lawns. The total nitrogen content of these grasses ranged from 1.7 percent to 2 percent of the dry matter. This percentage signifies again, large quantities of nutrient material that could be derived from vegetative litter. With respect to the amounts of nutrients applied and those removed in the cut dry matter, Ruelke and Prine (1971) observed that approximately 50 percent of the N, 88 percent of the P, and 79 percent of the K was recovered in the cut dry matter. Grasses will remove differing percentages of nutrients depending on the variety of the grass (Ruelke and Prine, 1968, 1971).

Such studies indicate the potential quantities of nutrients available from grass clippings or grass leaf litter. This is a brief review of the grass litter contribution from an urban environment, but deserves considerations of source control and urban land use management when discussing sources of pollution in urban runoff.

Summary

In concluding this section concerning litter-fall and its effects on urban runoff and stormwater, one is reminded of the immense complexity of the problem. It is evident that additional study is required. The present discussion only approaches the problem. Nevertheless, one can immediately recognize a potential problem to be considered that exists from natural litter-fall. In a forested area where this material becomes incorporated into the soil, a recycling of the organics and mineral nutrients takes place. Urbanization has a great impact on water quality and therefore requires serious land use refinements. McGriff (1972) has pointed out that urbanization increases the sediment load carried by streams, decreases ground water recharge, promotes eutrophication, and causes temperature variation in streams, all of which tend to alter water quality. Tree litter fall becomes a problem when it is remembered that several tons/acre of this material are deposited annually with perhaps 100 pounds (45 kg) of nitrogen entering the runoff of the area. Tree-shaded streets are a source of civic pride; however, more refined land use systems should be investigated to eliminate a serious water pollution and management condition from existing.

This section has been an attempt to characterize litter-fall by quantity and quality, as it might apply to urban water quality. The review of the literature warrants future research and consideration on this subject.

D. TREATMENT MODEL REFINEMENTS

Objectives and Background

The objectives of this portion of research were to:

1) add to the treatment model those treatment options which have developed subsequent to the original report;

2) add to the treatment model those treatment options which were considered infeasible at the time of original development;

3) review and refine the treatment options in the original model based on simulation of real data.

At the time of development of the original Model, biological treatment of storm flows was precluded due to the difficulties anticipated in maintaining large quantities of biologically active sludge needed for the treatment. As a result, it is difficult to simulate a real event when the associated dry weather treatment facility comprises biological treatment. Such is the case in the City of Lancaster where both North and South sewage treatment plants provide secondary biological treatment. In addition, the use of biological treatment for treating storm flows is currently being demonstrated in Kenosha Wisconsin (Agnew, 1973). It was, therefore, considered necessary to add this option to the SWMM in order to apply it to Lancaster and to extend the usefulness of the Model. Furthermore, new treatment devices such as the swirl concentrator (APWA, 1972) and high-rate disinfection (Glover, 1972) appear quite promising. The Revised Treatment Model is shown in Figure 7 and incorporates the above devices as well as others.

Recent Developments

Swirl Concentrator (Treatment Option 13)--This treatment device offers a high degree of performance in reducing the amount of settleable solids contained in combined sewer flows. The device consists of a circular basin in which rotary motion of the flow is induced by the kinetic energy of the incoming flow. This rotary motion causes the flow to follow a long spiral path through the circular chamber allowing the settleable solids to settle rapidly and be withdrawn from the bottom for subsequent treatment.

Modeling of this treatment unit is based on the work of APWA (1972) which utilized mathematical analyses and laboratory scale studies for design criteria. A full-scale, 36 foot (11m) diameter unit, is proposed for construction at Lancaster, Pennsylvania.

Data from the APWA study have been simplified and plotted in Figure 8. The unit has been modeled in such a way that given the flow, the size of the swirl concentrator, the particle sizes and specific gravities, and the fraction of particles of each size, the efficiency of suspended solids removal can be computed.

The Model performs the following steps:

1) Particle settling velocity, V_s, is determined by an iterative procedure. Initially, assumed velocity is computed using Stokes' law:

$$V_s = \frac{g}{18}(S_s - 1)\frac{d^2}{\nu} \qquad (7)$$

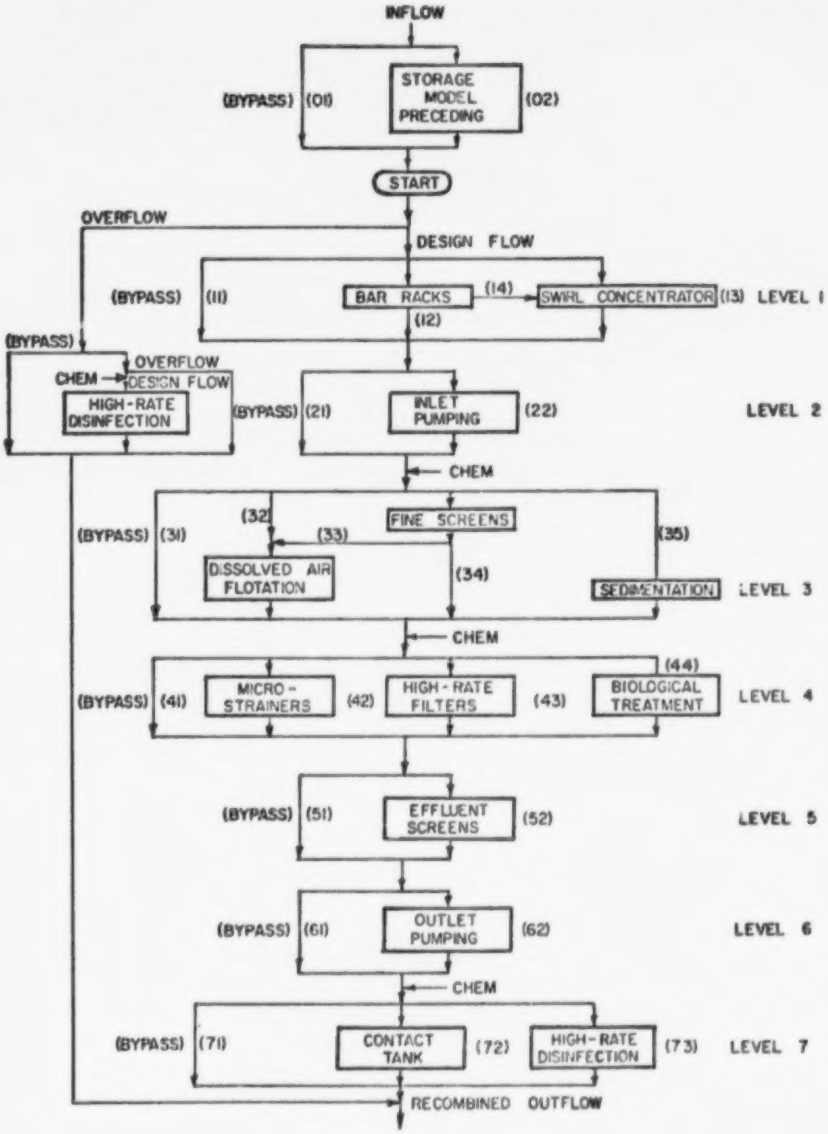
INFLOW
(BYPASS)
(01)
STORAGE MODEL PRECEDING
(02)
START
OVERFLOW
DESIGN FLOW
(BYPASS)
(11)
BAR RACKS
(14)
SWIRL CONCENTRATOR
(13)
LEVEL 1
(12)
(BYPASS)
OVERFLOW
CHEM
DESIGN FLOW
HIGH-RATE DISINFECTION
(BYPASS)
(21)
INLET PUMPING
(22)
LEVEL 2
CHEM
(32)
FINE SCREENS
(BYPASS)
(31)
(33)
(35)
DISSOLVED AIR FLOTATION
(34)
SEDIMENTATION
LEVEL 3
CHEM
(44)
(BYPASS)
(41)
MICRO-STRAINERS
(42)
HIGH-RATE FILTERS
(43)
BIOLOGICAL TREATMENT
LEVEL 4
(BYPASS)
(51)
EFFLUENT SCREENS
(52)
LEVEL 5
(BYPASS)
(61)
OUTLET PUMPING
(62)
LEVEL 6
CHEM
(BYPASS)
(71)
CONTACT TANK
(72)
HIGH-RATE DISINFECTION
(73)
LEVEL 7
RECOMBINED OUTFLOW

FIGURE 7.
Options Available in Revised Treatment Model.

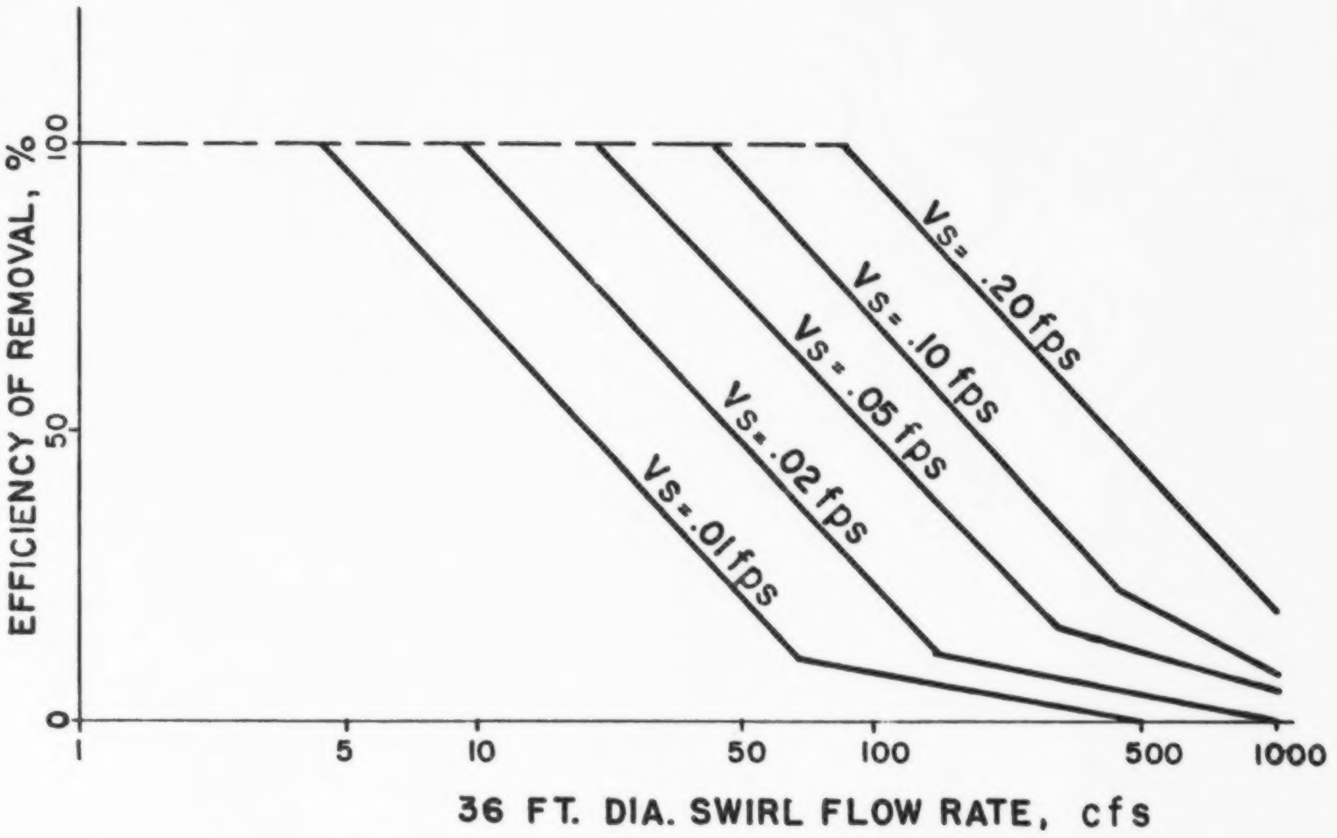

FIGURE 8. Swirl concentrator removal efficiency for settleable solids as a function of inflow and settling velocity, V_s (APWA, 1972).

where g = gravitational acceleration, cm/sec^2

ν = kinematic viscosity of water, cm^2/sec

S_s = specific gravity of the particle

d = diameter of the particle, cm

Using this V_s, the Reynolds Number, R, and the drag coefficient, C_D, are computed by the equations:

$$R = \frac{V_s d}{\nu} \tag{8a}$$

$$C_D = \frac{24}{R} + \frac{3}{\sqrt{R}} + 0.34 \tag{8b}$$

The particle settling velocity is then computed again using the following equation:

$$V_s = \sqrt{\frac{4}{3}\frac{g}{C_D}(S_s - 1)d} \quad \text{(cm/sec)} \tag{9}$$

Equations (8) and (9) are then used iteratively until the values of V_s used in Equation (8a) and determined in Equation (9) agree to within 0.01 cm/sec.

2) In order to use the curves developed in the APWA report, it is necessary to convert the particle settling velocity for the design swirl to prototype (36 ft diameter) settling velocity by applying the Froude scaling correction factor as follows:

$$V_{s\ proto}\ \text{(ft/sec)} = \frac{V_s}{30.48}\left(\frac{36}{D}\right)^{1/2} \tag{10}$$

where D is the design swirl concentrator diameter in feet and V_s is in cm/sec and computed above in Step 1.

3) Inflow into the prototype (36 ft diameter) swirl, Q_{proto}, is computed from inflow into the design swirl, Q_{design}, by Froude scaling:

$$Q_{proto} = Q_{design}\left(\frac{36}{D}\right)^{5/2} \tag{11}$$

4) The program then uses the family of curves shown in Figure 8 to compute the efficiency of removal. This figure is a linear approximation of Figure 17, Appendix 2, in the APWA report (1972).

5) The percent removal is then multiplied by the fraction of particles in that size range to give the total percent removal for that particle size.

6) This procedure is repeated for each particle size to get the total percent removal. Settling velocities shown in Figure 8 are for settleable solids. It is *estimated* that percent removal of suspended solids will be 60% of that for settleable solids. Hence, suspended solids removal is taken as 60% of the value computed in Steps 4 and 5. Improvements in these assumptions will be made upon receipt of data from actual installations.

7) As modeled by the SWMM, BOD consists of a contribution due to suspended organic solids as well as the dissolved amount. Hence, BOD removal is assumed at 0.4 times the suspended solids removal, except that if incoming BOD is less than that computed in this manner, BOD removed will be taken as 0.35 times BOD input.

Sedimentation Revisions (Treatment Option 35)--Efficiency of suspended solids removal by primary sedimentation is computed in the original model using the following relationship (EPA, 1972a):

$$\eta = 0.656 + \frac{\text{SS Conc. x .06}}{190} - 0.40\left(\frac{\text{OVFRA} - 300}{2000}\right) \qquad (12)$$

where OVFRA is the overflow rate in gpd/sq ft and SS concentration has units of ppm (mg/l).

In order to check the validity of this relationship, several data points obtained from Eckenfelder and McCabe (1956) and Metcalf and Eddy (1935) are plotted on Figure 9. The curve represented by the above relationship is plotted on the same figure for suspended solids concentration of 400 ppm along with a sedimentation curve as developed by Smith (1968) based on the following relationship:

$$\eta = 0.82e^{-\text{OVFRA}/2780} \qquad (13)$$

where OVFRA is equal to overflow rate in gpd/sq ft. Figure 9 shows that Smith's curve gives a better representation of the settling efficiency than the one computed by the equation in the original SWMM. The Model has, therefore, been revised to incorporate Smith's equation.

Biological Treatment (Treatment Option 44)--Biological treatment as defined herein is taken as the aeration and secondary settling portion of the treatment. Consequently, it is necessary to use treatment option 35 in conjunction with the treatment option 44. The efficiency of suspended solids and BOD reduction in treatment plants using sedimentation and biological treatment exceeds 90 percent for flow up to a design flow. The modeling, therefore, is formulated in such a way that overall efficiency of SS and BOD removal will be close to 90 percent when both treatment options 35 and 44 are provided. When the inflow to the unit exceeds the design flow, the excess flow is bypassed. A sample output from secondary treatment is given at the end of this sub-section--*Revised Treatment Output.*

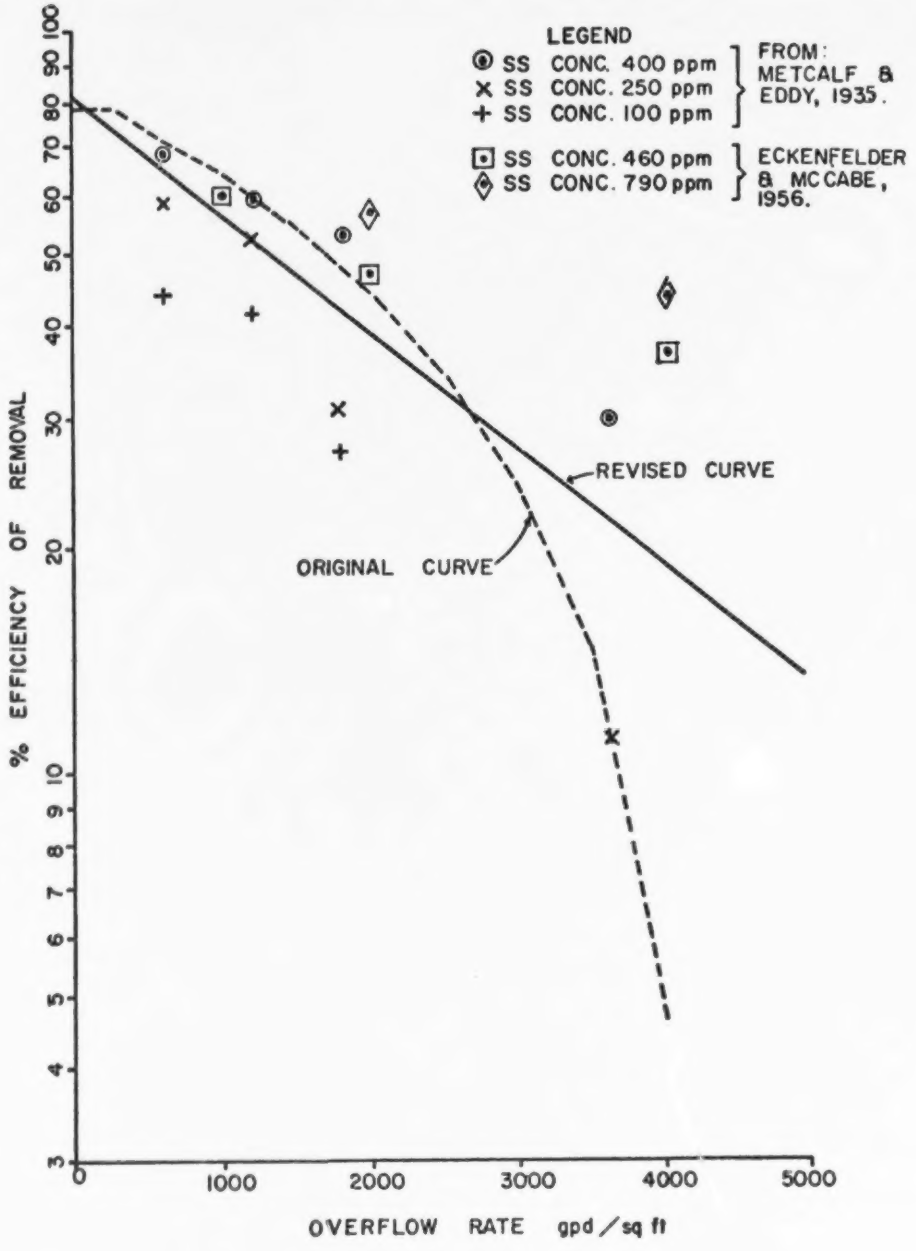

FIGURE 9.
Removal Efficiency by Sedimentation. Original and Revised Relationships.
Compared with Data of Eckenfelder and McCabe (1956)
and Metcalf and Eddy (1935).

High Rate Disinfection (Treatment Option 73)--The modeling of this unit is based on the work of Glover (1972). This treatment option is similar to chlorination (treatment option 72) with the following exceptions:

1) The volume of the tank is selected to provide a two-minute detention period at design flow.

2) The chlorine demand is taken at not less than 5 ppm and not more than 10 ppm.

The unit will provide a BOD reduction computed at 2.0 times the chlorine demand but not more than 50 percent of incoming BOD. Coliform reduction is computed in subroutine KILL in a manner similar to option 72. This unit can also be used as a sole disinfection device on the overflow.

Other Treatment Options--Modeling of other treatment options shown in Figure 7 remains the same as formulated in the original SWMM. It is anticipated that other revisions may be made when new data become available. In addition, there are several other treatment methods currently under investigation such as:

1) rotating biological contactors;

2) high-rate plastic media trickling filters;

3) tube settlers;

4) powdered and activated carbon absorption; and

5) chemical treatment.

Revised Treatment Output-- The following example illustrates the modeling of primary and secondary treatment given the combined sewage passing through the North Treatment Plant, Lancaster, Pennsylvania. Figures 10, 11, and 12 show the time-varying inflows of combined sewage, BOD, and suspended solids to the treatment facility. Tables 18 and 19 illustrate treatment model input. The treatment scheme selected (see Figure 7 and Table 19) includes the *revised* sedimentation option (treatment option 35) and the *new* biological treatment option (treatment option 44):

1) the storage model is bypassed at level 0;

2) the waste water is passed through bar racks (treatment option 12) at level 1;

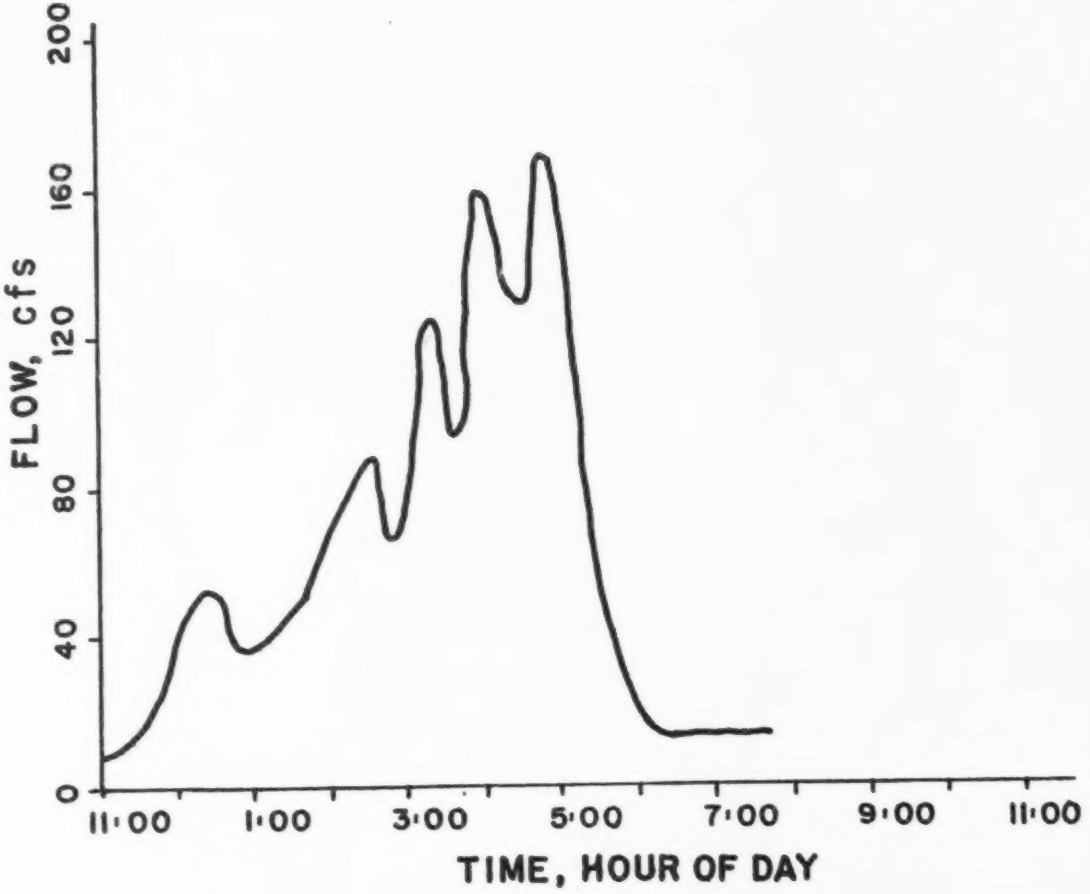

FIGURE 10. Combined Inflow to North Treatment Plant Upstream of Diversion Chamber.

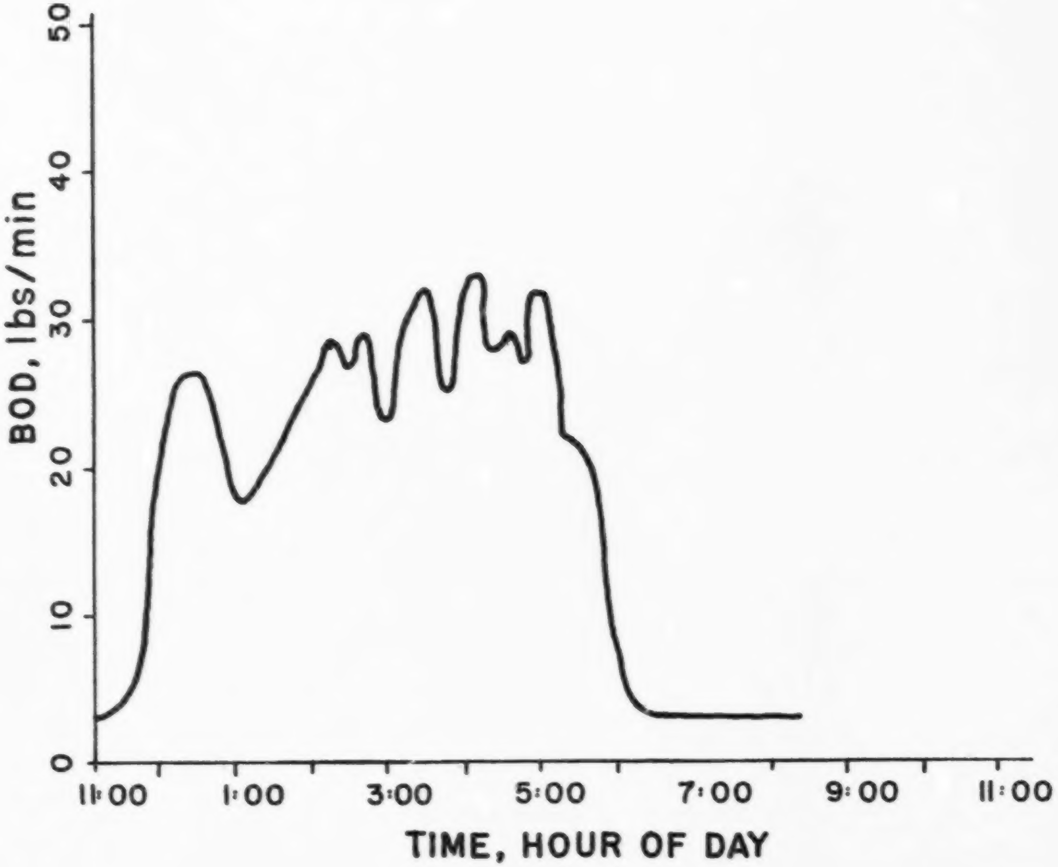

FIGURE 11. Combined BOD Load to North Treatment Plant Upstream of Diversion Chamber.

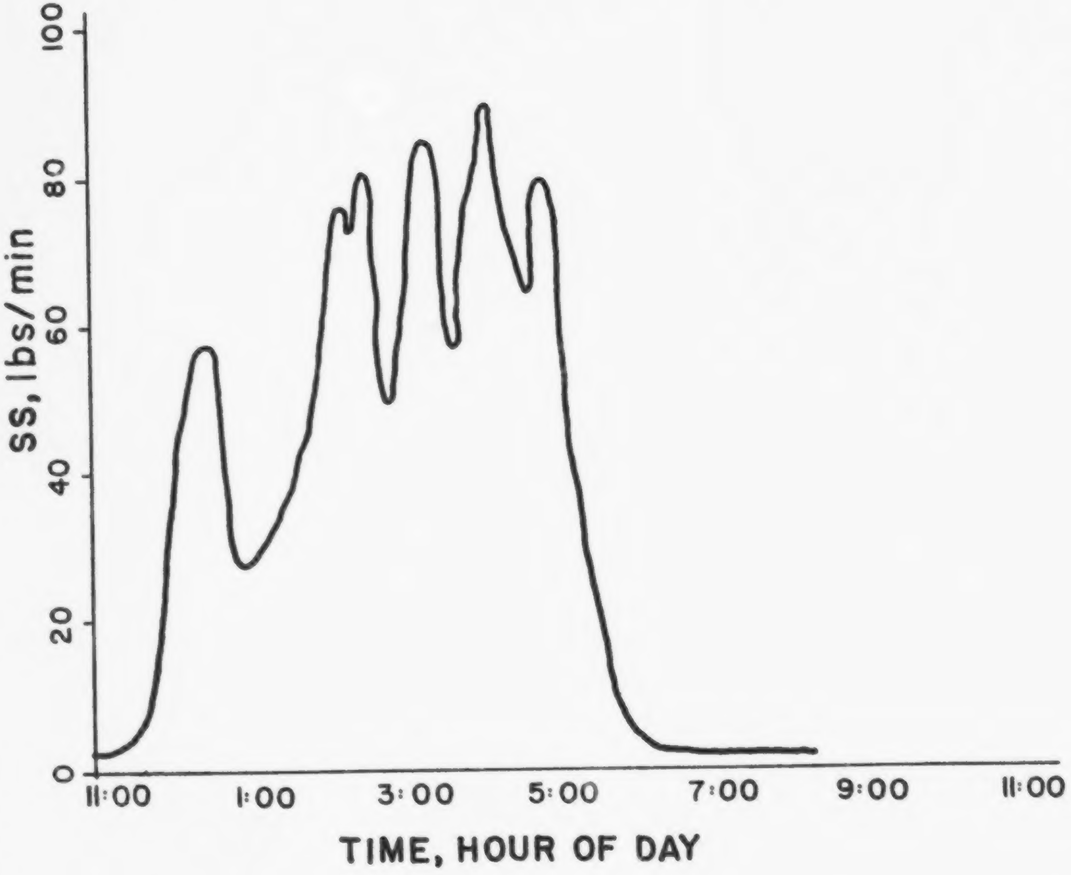

FIGURE 12. Combined Suspended Solids Load to North Treatment Plant Upstream of Diversion Chamber.

```
        UPDATED BY UNIVERSITY OF FLORIDA

        DECEMBER 1972


        STORAGE  PROGRAM CALLED



    LANCASTER  PENNSYLVANIA  NORTH  DRAINAGE  DISTRICT
OUTPUT FROM EXTERNAL STORAGE/TREATMENT MODELS

                  TRANSPORT MODEL OUTFALLS AT THE FOLLOWING ELEMENT NUMBERS:
                            1

                  INPUT TO TREATMENT MODEL SUPPLIED FROM TRANSPORT MODEL EXTERNAL ELEMENT NUMBER  1


NUMBER OF RUNS            =        1

TIME-STEP SIZE            =     5.00 MIN.

NO. TIME-STEPS MODELED    =      144

TRIBUTARY AREA            =  1014.00 ACRES

NO. TRANSP. MOD. OUTFALLS =        1

NO. OF POLLUTANTS         =        3

TIME ZERO                 =  39600.0 SEC
```

Table 18. BEGINNING OF STORAGE/TREATMENT.

```
-------  RUN NO. 1  -------
INPUT DATA FOR TREATMENT PACKAGE FOLLOWS


                          CHARACTISTICS OF THE TREATMENT PACKAGE ARE

                              LEVEL      MODE          PROCESS


                                0         01        NO SEP. STORAGE
                                1         12           BAR RACKS
                                2         21                             (BYPASS)
                                3         35         SEDIMENTATION
                                4         44        BIOLOGICAL TREAT
                                5         51                             (BYPASS)
                                6         61                             (BYPASS)
                                7         72          CONTACT TANK

**** WARNING ****

      THE FOLLOWING COMBINATIONS OF TREATMENT OPTIONS ARE CONSIDERED ECONOMICALLY INADVISEABLE - SIMULATION CONTINUES

    ITREAT    WITH    ITREAT

      72      WITH      35

                                                                      IPRINT = 2.  ICOST = 3.  IRANGE = 0.   ITABLE = 0

   SPECIFIED TREATMENT CAPACITY USED.

           DESIGN FLOWRATE =     15.47 CFS.
           TREATMENT SYSTEM INCLUDES MODULE UNITS
                DESIGN FLOW IS THEREFORE INCREASED TO NEXT LARGEST MODULE SIZE
                ADJUSTED DESIGN FLOWRATE =     15.47 CFS.. =     10.00 MGD.
                (KMGD =  2)

           NO STORAGE FROM A SEPARATE STORAGE MODEL IS ASSOCIATED WITH THIS TREATMENT MODEL

           PRELIMINARY TREATMENT BY MECHANICALLY CLEANED BAR RACKS  (LEVEL 1)
                NUMBER OF SCREENS    =       2
                CAPACITY PER SCREEN =       7.73 CFS
                SUBMERGED AREA       =       2.50 SQ.FT. (PERPENDICULAR TO THE FLOW)
                FACE AREA OF BARS    =       3.61 SQ.FT.

           INFLOW BY GRAVITY (NO PUMPING)  (LEVEL 2)

           TREATMENT BY SEDIMENTATION (LEVEL 3) - (NO ASSOCIATED STORAGE)
                DESIGN OVERFLOW RATE =   1240.00 GPD/SQ.FT.  (1600 SUGGESTED)
                SED TANK DEPTH        =     10.00 FEET       (8 FEET SUGGESTED)
                NUMBER OF SED TANKS  =          1
                SURFACE AREA          =   8064.51 SQ.FT./TANK
                NO CHLORINE ADDED

           BIOLOGICAL TREATMENT  (LEVEL 4)

           NO EFFLUENT SCREENS  (LEVEL 5)
```

Table 19. TREATMENT MODEL INPUT.

3) level 2 is bypassed;

4) primary treatment is completed in the form of sedimentation at level 3;

5) secondary treatment is provided by biological methods (activated sludge aeration and secondary settling) at level 4;

6) level 5 is bypassed;

7) level 6 is bypassed;

8) chlorination is the final treatment given in the contact tanks at level 7.

Table 20 illustrates the effectiveness of the treatment scheme selected. Approximately 90 percent of the input BOD, 92 percent of the initial suspended solids concentration, and 99.99 percent of the initial coliform (MPN) concentration are removed. Of course, the treatment options at levels 3 and 4 account for the bulk of the treatment efficiency.

E. RECEIVING MODEL REFINEMENTS

The Receiving Model has proven to be quite versatile in the applicability to different problems, and has been used at the University of Florida for purposes other than to model the effect of stormwater runoff on a receiving water system. For example, it has been applied to a lake that receives input from adjacent agricultural areas (Perez *et al.*, 1972; Perez, 1972) and to a marsh system (Heaney and Huber, 1972). The latter application, in particular, prompted three modifications.

Firstly, the spatial variation of rainfall can be of considerable importance when modeling a large marsh system. Revisions were made to read in records from as many as 50 rain gages. Rainfall from each gage is then apportioned among the various junctions of the Receiving Model network. For example, 40 percent of the rainfall for, say, junction 1 might be assigned from gage 5, 35 percent from gage 6 and 25 percent from gage 7 in a hypothetical situation. The total rainfall at each junction can consist of contributions from one or more point rainfall measurements. The percentages are determined by overlaying the Thiessen polygon networks determined by the Receiving Model junctions and by the rain gage array. Percentages are then assigned on the basis of the amount of junction area contributed by Thiessen polygons of the different rain gages.

```
SUMMARY OF TREATMENT EFFECTIVENESS

       TOTALS                    FLOW (M.G.)     BOD (LB)     SS (LB) COLIF (MPN)
         INPUT                      14.946        22675.9     33425.1    1.17E 13
         OVERFLOW (BYPASS)          10.703         9637.7     19391.4    9.15E 12
         TREATED                     4.243        13034.2     14033.7    2.58E 12
         REMOVED                     0.044        11752.0     12853.3    2.58E 12
         RELEASED                   14.902        10323.8     20571.7    9.15E 12


       REMOVALS                  FLOW(M.G.)      BOD (LB)     SS (LB)
         LEVEL 1                     0.000            9.5       190.9                    =    BAR RACKS
         LEVEL 3 (TOTAL)             0.032         4202.8      7940.5                    =  SEDIMENTATION
         LEVEL 4                     0.011         7660.7      4721.8                    = BIOLOGICAL TREAT
         LEVEL 5                     0.0              0.0         0.0                    = NO EFFL. SCREENS
         LEVEL 7                     0.0            479.0         0.0                    =   CONTACT TANK
           TRASH:
         BAR RACKS                  25.449 CU.FT (AT 50 LB/CU.FT.)
         EFFLUENT SCREENS            0.0   CU.FT (AT 50 LB/CU.FT.)


       REMOVAL PERCENTAGES       FLOW (VOL)      BOD (LB)     SS (LB) COLIF (MPN)
         OF OVERALL INPUTS           0.29           53.23       38.45       22.00
         OF TREATED FRACTIONS        1.03           90.14       91.59       99.99


       CONSUMPTIONS (LB)           CHLORINE      POLYMERS
         LEVEL 3                     0.0            0.0                                  =  SEDIMENTATION
         LEVEL 4                     0.0            0.0                                  = BIOLOGICAL TREAT
         LEVEL 7                   261.9            0.0                                  =   CONTACT TANK
         TOTAL                     261.9            0.0

    REPRESENTATIVE VARIATION OF TREATMENT PERFORMANCE WITH TIME (OVERALL).

TIME                   11:20     12:30     13:40     14:50     16: 0     17:10     18:20     19:30     20:40     21:50     23: 0
WATER
   AV. FLOW (CFS)       9.45     39.07     42.31     85.34     96.74    150.01     21.19     10.47      9.78      9.62      9.56
BOD
   ARRIVING (MG/L)    815.11    269.77    203.35    107.46     83.08     59.64    289.25    720.49    786.76    801.19    808.32
   RELEASED (MG/L)     86.67    173.96    135.52     88.60     70.73     53.88    100.38     77.61     83.98     85.34     85.82
   % REDUCTION (LB)    89.51     35.66     33.47     17.55     14.85      9.58     65.46     89.35     89.46     89.49     85.49
S. SOLIDS
   ARRIVING (MG/L)    680.03    386.21    312.41    297.04    234.53    140.59    263.87    598.45    653.50    665.55    665.83
   RELEASED (MG/L)     51.56    248.42    209.35    248.27    174.83    127.33     90.09     47.34     50.16     50.75     50.95
   % REDUCTION (LB)    92.52     35.82     33.10     16.43     14.50      9.37     66.02     92.18     92.42     92.47     92.49
COLIFORMS
   ARR (MPN/100ML)  4.08E 03 2.91E 04 2.40E 04 3.40E 04 2.24E 04 1.59E 04 5.42E 03 3.86E 03 3.95E 03 4.02E 03 4.04E 03
   REL (MPN/100ML)  3.09E-01 1.77E 04 1.53E 04 2.79E 04 1.92E 04 1.42E 04 1.48E 03 2.89E-01 3.03E-01 3.06E-01 3.07E-01
   % REDUCTION (LB)    99.99     39.51     36.51     18.11     15.98     10.31     72.78     99.99     99.99     99.99     99.99

    OUTFLOW BY GRAVITY (NO PUMPING)  (LEVEL 6)

    TREATMENT BY CHLORINE CONTACT TANK (LEVEL 7)
          NUMBER OF DOSING UNITS =             2
          DOSING RATE PER UNIT   =   2000.00 LB/DAY
          MAXIMUM DEMAND RATE    =   2086.17 LB/DAY
          VOLUME OF CONTACT TANK =    13923. CU.FT. AT 15 MIN. DETENTION TIME
```

Table 20. TREATMENT MODEL OUTPUT.

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Secondly, areas with high vegetation, such as flood plains and marshes, exhibit a variation of Manning's roughness, n, with flow depth. For example, measurements made by the Corps of Engineers (1956) in a sawgrass marsh in the Florida Everglades conservation areas may be used to account for this effect. The program allows a user-specified depth vs n variation.

Thirdly, overall hydraulic characteristics of marshes that contain channels change when the channels overflow. The channels have low roughness and permit high-flow rates but have negligible available storage compared to the adjacent overland flow regime in the marsh. However, the latter regime is subject to high roughness, typically small slopes and low flow rates. Hence, the concept of parallel channels between two junctions was introduced to allow one channel to simulate the actual channel and a parallel channel to simulate the overland flow on the marsh. Each channel can thus have differing hydraulic characteristics while transporting water between the same two points.

A modification that has resulted in increased flexibility in the use of the Receiving Water block has been the incorporation of the O'Connor-Dobbins formula to compute reaeration coefficients and incorporation of a linear relationship to compute O_2 saturation concentrations as a function of chloride concentration based on data given by Clark, Viessman and Hammer, (1971). Included in the overall modification is the ability to vary oxygen saturation values and/or reaeration coefficients through the use of three options:

1) a constant value at all junctions may be used (as before), supplied by the user;

2) a different constant value may be read in for each junction;

3) the value may be computed at each junction at each time step.

Clearly, chlorides must be modeled when the third option is used to compute O_2 saturations. This feature computes considerably more realistic saturation values when simulating estuaries, as long as the chloride distribution is adequately modeled.

The third option allows computation of the reaeration coefficient, K_2, at each junction at each time-step using the O'Connor-Dobbins (1958) formula:

$$K_2 = \frac{\sqrt{D_M U}}{H^{3/2}} \qquad (14)$$

where K_2 = reaeration coefficient, sec^{-1}

D_M = molecular diffusivity of O_2 in water (a function of temperature), ft^2/sec

U = average velocity at a junction, ft/sec

H = junction depth, ft.

Use of this formula allows more accurate K_2 values in areas of changing flow conditions.

F. NETWORK AGGREGATION TECHNIQUE

In order to add the capability for modeling larger areas, the Combine Block has been added to the Storm Water Management Model. This block has two main objectives. The first is to collate two or more different output data-sets from Runoff, Transport, Storage/Treatment, or any combination thereof. This new data-set could then be used as input into any block (Transport, Storage/Treatment or Receiving Water), except Runoff. For example (Figure 13), an output data-set from Transport area 'A' with manhole numbers 5, 6, 12 was collated with an output data-set from Transport area 'B' with manhole numbers 1, 3, 6, 19. Manhole number 6 is common between both output data-sets, therefore the hydrographs and pollutographs from both manholes are added together. The new output data-set produced from the Combine Block has manhole numbers 1, 3, 5, 6, 12, 19. This new data-set could then be used as input to either the Transport, Storage/Treatment, or Receiving Water Blocks. The second objective is to combine different data-sets and manholes into a single data set with one manhole. For example (Figure 14), an output data-set from Transport area 'Y' with manhole number 23 is to be used as input into the Receiving Water Block junction* number 14. The Combine portion of the Combine Block would be used to combine the two output data-sets into one data-set with one manhole. This manhole number would correspond to the junction number of the Receiving Water Block.

G. MONITORING ROUTINE FOR DETERMINATION OF LARGEST SOURCES OF FLOW AND POLLUTANTS

In order to determine the location and feasibility of various pollutant source control alternatives and runoff storage devices, information in the system should be analyzed. To aid in this analysis, a tabular printout of the total BOD, suspended solids, and flow entering each inlet is summarized for each SWMM run as shown in Table 21. Before discussing the applications for these summary data, a brief description of the table will be given.

* Junction number and manhole number are synonymous.

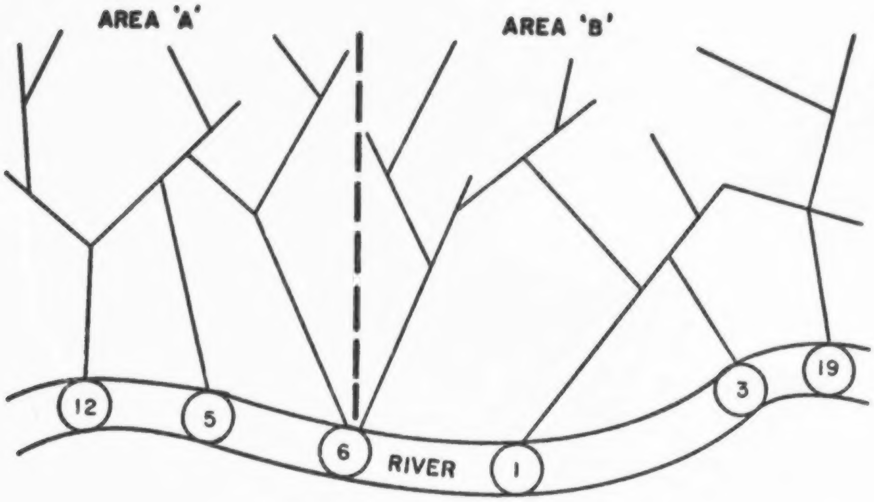
AREA 'A'
AREA 'B'
12
5
6
RIVER
1
3
19

FIGURE 13.
Hypothetical Drainage Network A.

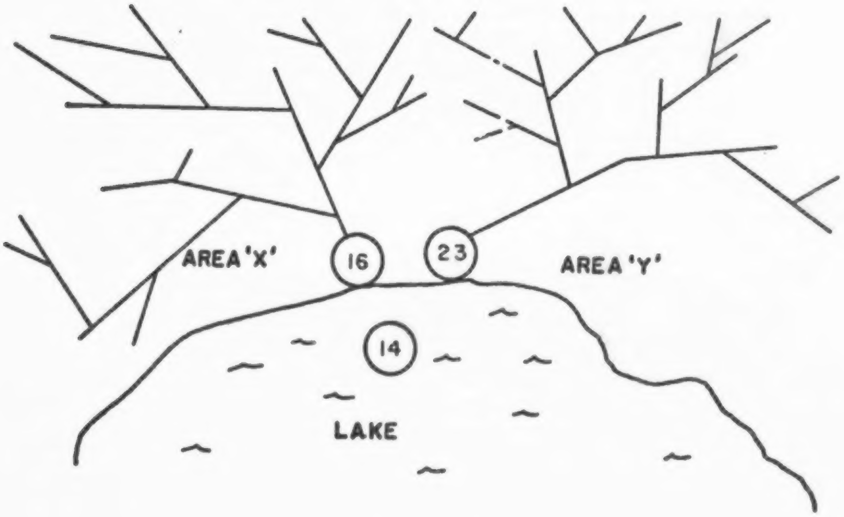
AREA 'X'
16
23
AREA 'Y'
14
LAKE

FIGURE 14.
Hypothetical Drainage Network B.

TABLE 21.
PRINTOUT FROM POLLUTANT MONITORING ROUTINE

```
RESULTS OF POLLUTANT MONITORING ROUTINE

POLLUTANTS ASSOCIATED WITH MANHOLES (INLET POINTS) RANKED IN ORDER OF SIGNIFICANCE OF SUSPENDED SOLIDS.

                  SUSPENDED  SOLIDS (LB)                5 - DAY BOD (LB)
                  -------------------------             ----------------         TOTAL
RANK  INLET  RUNOFF     C.B.F.  F.SCOUR  TOTAL SS    RUNOFF  C.B.F.  TOTAL BOD   INFLOW (CF)

  1    31   .2828E 04   13.59   0.08   .2828E 04      3.51   20.63    24.14      .1038E 03
  2    82   .3448E 04   22.67   0.01   .3478E 04      2.56   24.01    26.57      .2408E 02
  3    35   .2545E 04   34.68   0.00   .2578E 04      2.69   52.32    55.01      .5558E 02
  4    46   .1088E 04    5.23   0.0    .1088E 04      0.44    7.85     8.29      .7318E 01
  5    79   .5545E 02   15.24   0.04   .1158E 03      5.78   22.87    28.65      .3215E 02
  6    68   .8088E 02   15.82   0.0    .1008E 03      5.16   25.72    30.88      .4585E 02
  7    37   .3503E 02   37.14   0.02   .7229E 02      2.54   49.70    52.24      .3318E 02
  8    43   .4356E 02   24.42   0.02   .6878E 02      2.84   32.62    35.46      .4058E 02
  9    76   .3556E 02   24.46   0.0    .6043E 02      2.83   32.62    35.45      .5478E 02
 10    40   .3258E 02   22.67   0.03   .5525E 02      2.17   34.01    36.18      .3088E 02
 11    50   .3865E 02   13.59   0.13   .5240E 02      2.89   20.63    23.81      .5548E 02
 12    45   .3048E 02   20.63   0.0    .5113E 02      2.01   31.39    33.41      .3148E 02
 13    55   .3838E 02   13.59   0.01   .5077E 02      2.73   20.93    23.66      .6118E 02
 14    54   .3818E 02   12.21   0.04   .4048E 02      2.02   18.31    20.33      .2308E 02
 15    58   .2715E 02   13.59   0.0    .2711E 02      1.78   20.93    22.71      .5558E 02
 16    62   .2729E 02    8.72   0.08   .3601E 02      1.93   13.08    15.01      .4038E 02
 17    46   .2205E 02   13.59   0.07   .3618E 02      1.47   20.93    22.40      .2458E 02
 18    52   .2028E 02    5.23   0.09   .2558E 02      2.18    7.85    10.03      .2395E 02
 19    97   .1655E 02   17.44   0.12   .3405E 02      1.33   26.16    27.49      .2028E 02
 20    38   .1574E 02   13.59   0.0    .2578E 02      1.04   20.53    21.59      .1518E 02
 21    75   .2045E 02    6.58   0.05   .2738E 02      1.31   10.46    11.77      .1088E 02
 22    67   .2258E 02    3.46   0.23   .2688E 02      1.42    5.23     6.65      .1518E 02
 23    56   .2098E 02    5.23   0.0    .2408E 02      1.28    7.85     9.12      .1348E 02
 24    80   .7803E 01   15.70   0.03   .2358E 02      0.53   23.54    24.07      .1018E 02
 25    71   .1819E 02    3.46   0.00   .2168E 02      1.14    5.23     6.37      .7238E 01
 26    80   .1251E 02    8.72   0.01   .2128E 02      0.77   12.08    12.85      .6568E 01
 27    53   .1255E 02    5.23   0.02   .1888E 02      0.93    7.85     8.78      .9588E 01
 28    47   .1369E 02    3.49   0.0    .1718E 02      0.93    5.23     6.17      .1388E 02
 29    83   .1265E 02    3.46   0.0    .1618E 02      0.96    5.23     6.19      .2028E 02
 30    81   .1058E 02    5.23   0.03   .1581E 02      0.72    7.85     8.57      .1218E 02
 31    58   .6305E 01    8.72   0.0    .1508E 02      0.46   13.08    13.54      .1205E 02
 32    87   .5432E 01    1.74   0.01   .1128E 02      0.60    2.62     3.22      .9028E 01
 33    61   .5575E 01    3.46   0.01   .5478E 01      0.36    5.23     5.59      .3398E 01
 34    64   .3475E 01    5.23   0.0    .8518E 01      0.34    7.85     8.19      .1158E 02
 35    65   .5255E 01    2.45   0.0    .8748E 01      0.47    5.23     5.70      .2518E 02
 36    80   .3358E 01    3.49   0.06   .6848E 01      0.29    5.23     5.52      .2088E 01
 37    53   .3155E 01    3.46   0.0    .6648E 01      0.30    5.23     5.53      .8278E 01
 38    69   .4085E 01    1.74   0.0    .5808E 01      0.25    2.62     2.87      .2568E 01
 39    63   .1798E 00    5.23   0.10   .5518E 01      0.14    7.41     7.55      .2098E 01
 40    51   .2635E 01    1.74   0.0    .4378E 01      0.20    2.62     2.82      .3528E 01
 41    99   .4208E 01    0.0    0.0    .4208E 01      0.20    0.0      0.20      .6108E 01
 42    20   .0            0.0    0.15   .1508E 00      0.0     0.0      0.0       .7328E 00
 43    24   .0            0.0    0.00   .2418E-02      0.0     0.0      0.0       .3668E 00
 44     7   .0            0.0    0.0    .0             0.0     0.0      0.0       .0
 45    18   .0            0.0    0.0    .0             0.0     0.0      0.0       .2478E 00
 46    19   .0            0.0    0.0    .0             0.0     0.0      0.0       .1568E 00
 47    20   .0            0.0    0.0    .0             0.0     0.0      0.0       .3378E 00
 48    29   .0            0.0    0.0    .0             0.0     0.0      0.0       .3328E 00
```

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The first column labeled *Rank* indicates that the inlet numbers in column 2 are listed in order of magnitude of suspended solids entering the system at that inlet. The first inlet listed collects the greatest amount of solids and the last inlet the least amount, as shown under the column labeled *Total SS*. Columns 3 through 5 give a breakdown of the total pounds of suspended solids entering the system at each inlet by surface runoff, dry weather flow (DWF), and pipe scour. The pipe scour is from the adjacent upstream pipes. Columns 7 thru 8 list the pounds of 5-day BOD entering each inlet from runoff (including catch-basins) and dry weather flow, respectively. The total BOD runoff and dry weather flow is given for each inlet under the *Total BOD* column. The last column labeled *Total Inflow* gives the volume of water entering each inlet from both surface runoff and dry weather flow during the storm event. The data listed in this printout, Table 21, represent a decomposition of the total drainage system into its localized inputs. All information pertaining to the time sequence of events has been eliminated since the data represent run totals.

In attempting to locate a pollution control device upstream from the drainage system's outlet, a decision between alternate locations will probably be required. Upstream storage tanks will often be located at points in the network where branches or inlets with disproportionately high flows enter the system. Similarly, any treatment alternate such as sedimentation tanks, catchbasin cleaning, and sewer flushing might be systematically applied at points in the drainage network which have significant pollutant contributions.

Transferring some of the information from Table 21 to a map of the Stevens Avenue drainage area, Figure 15 enables the analysis of the spatial arrangement of pollutant inputs into the sewer network. The ten most significant suspended solids inputs account for 94 percent of the total solids with the four eroding sites contributing more than 90 percent of the total. This indicates the possible significance of erosion-produced solids which can now be predicted by the SWMM (see Section IV-B). Highest BOD loads enter inlets in the two western branches of the network. These ten inlets account for 52 percent of the total BOD. Pipes in which significant amounts of deposited solids are scoured during the storm event are also indicated on Figure 15. These ten pipes contribute 68 percent of the total scoured solids but less than 1 percent of the total suspended solids. Where pipe scour is found to be a significant portion of the first flush effect, an analysis of this type can indicate a "best" operating policy for sewer flushing.

Analyzing a drainage network utilizing the data from Table 21 is at least a first step in evaluating the spatial distribution of flow and pollutant inputs. Data arranged in this manner become information which can be further investigated utilizing other modeling options.

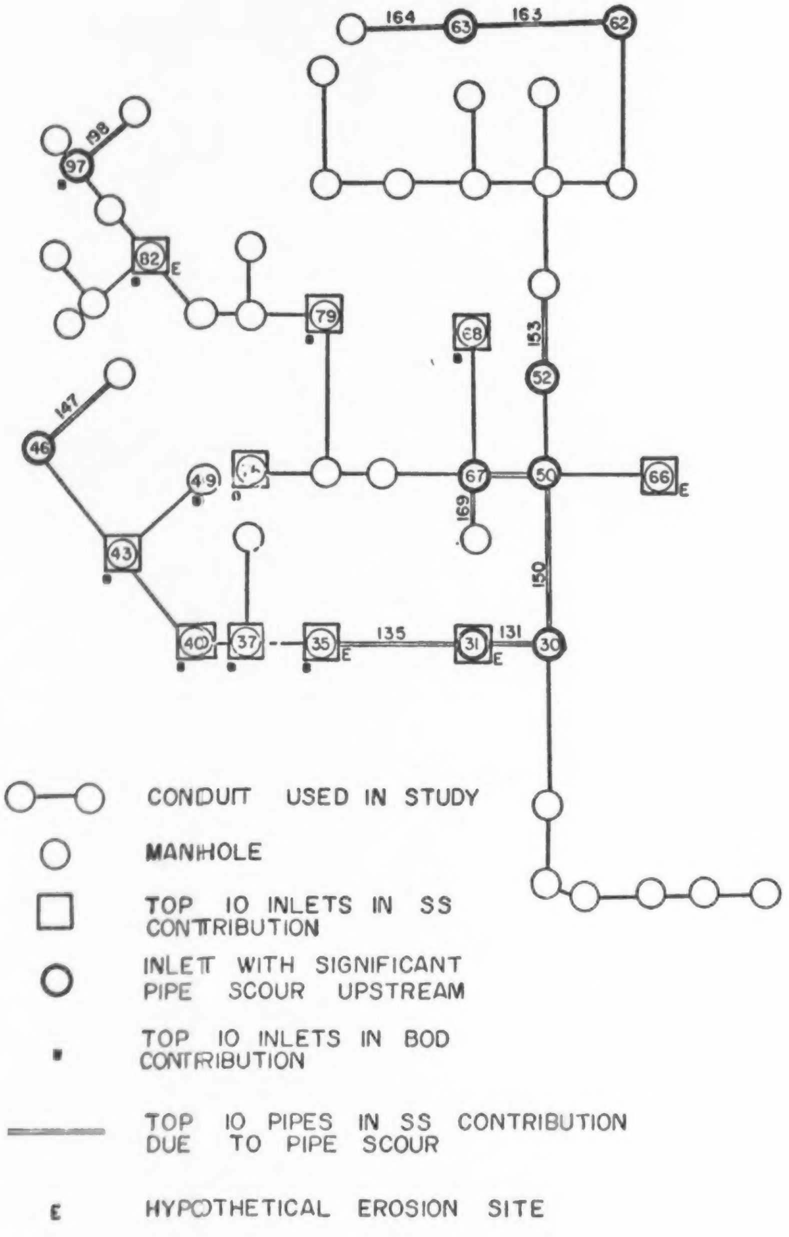

164
63
163
62
198
97
82
E
79
68
153
52
147
46
49
67
50
66
E
169
43
150
40
37
35
E
135
31
E
131
30
CONDUIT USED IN STUDY
MANHOLE
TOP 10 INLETS IN SS CONTRIBUTION
INLET WITH SIGNIFICANT PIPE SCOUR UPSTREAM
TOP 10 INLETS IN BOD CONTRIBUTION
TOP 10 PIPES IN SS CONTRIBUTION DUE TO PIPE SCOUR
E
HYPOTHETICAL EROSION SITE

FIGURE 15.
Spatial Analysis of Pollutant Inputs into the Stevens Avenue Drainage Network.

SECTION V

TESTING OF THE SWMM IN LANCASTER, PENNSYLVANIA

A. INTRODUCTION

The City of Lancaster, Pennsylvania, population 79,500, is situated in a drainage area of about 8.24 square miles (21.34 km^2). The receiving stream in the Lancaster area is the Conestoga River which drains an area of approximately 473 square miles (1225 km^2) into the Susquehanna River. The average flow is 387 cfs (11 m^3/sec) with a maximum recorded flow of 22,800 cfs (638 m^3/sec).

There are two sewage treatment plants within the City, both of which discharge into the Conestoga River. The North Plant with a capacity of 10 mgd (3.80 x 10^4 m^3/day) serves a population of 36,000 people, and the South Plant, recently expanded from 6 mgd (2.28 x 10^4 m^3/day) to 12 mgd (4.56 x 10^4 m^3/day) is designed to serve 69,000 people. Both plants provide secondary treatment. About one third of the flow to the North Plant is derived from areas with separate sewers outside the City serving an estimated population of 17,500 people and some industries. The remaining two thirds of the sewage flow to the North Plant is derived from the combined sewers serving the north part of the City plus about 250 suburban acres estimated to have 18,500 people and many water-using industries. In addition, most of the year the water table is high resulting in considerable infiltration. An overflow line diverts excess flow to the Conestoga during wet weather. The North Plant drainage area is estimated at 3.72 square miles (9.63 km^2).

The South Plant is designed to handle a population of 34,500 served by combined sewers and, in addition, up to an approximately equal amount from separated sewers throughout the surrounding area. The South Plant drainage area encompasses 4.52 square miles (11.71 km^2) and is comprised of four districts. Stevens Avenue district which is the subject of an EPA demonstration grant is one of the four districts connected to the South Plant. Three of the districts, including Stevens Avenue, pump the sewage from a receiving station within the district to the South Plant. All locations have overflow arrangements that discharge into the Conestoga River when the capacity of the system is exceeded.

The total drainage area of the Stevens Avenue district is 227 acres (92 na) which, while only about 4.3 percent of the total Lancaster drainage area served by North and South treatment plants, is 17 percent of the drainage area designed to flow into the South Plant from combined sewers. The population within the Stevens Avenue district is estimated at 3,900. Figure 16 illustrates various drainage districts within the City.

B. DEMONSTRATION GRANT DESCRIPTION

In order to remedy the situation resulting from combined sewer overflows, the City of Lancaster decided to explore means other than sewer separation.

Construction of several underground silos at various locations within the City is contemplated for retention of overflow during wet periods and subsequent pumping to the treatment plants during low flow periods.

Stevens Avenue district was selected as the demonstration site for evaluation of the effectiveness of a silo in combating combined sewer overflows. The sewer layout for Stevens Avenue district is shown in Figure 17. During normal dry weather periods, the dry weather flow is pumped to the South Treatment Plant. During wet periods, when the incoming flow to the pump station exceeds the capacity of the station, the overflow discharges directly into the Conestoga River through a 60-inch (1520 mm) sewer located at point 6 on Figure 16.

The City of Lancaster also authorized APWA to develop design parameters for a full-scale swirl concentrator for removal of solids prior to the retention of flow in the underground silo. Location of the demonstration site is shown in Figure 17. A flow diagram of the proposed swirl concentrator-silo treatment is presented in Figure 18. In order to fully evaluate this treatment, the city decided to include chlorination and microstraining as a part of this demonstration project. The capacity of the silo is expected to be 160,000 ft^3 (4480 m^3).

The tasks assigned to the University of Florida were as follows:

1) conduct further verification and testing of the Storm Water Management Model based on active overflow measurements on selected storm events and to make refinements to the Model;

2) provide results of simulations to the APWA in order for it to develop design criteria and sizing of the swirl concentrator;

3) simulate the effect of the swirl concentrator-underground silo treatment; and

4) simulate the effect of combined sewer overflow from the entire city to the Conestoga River.

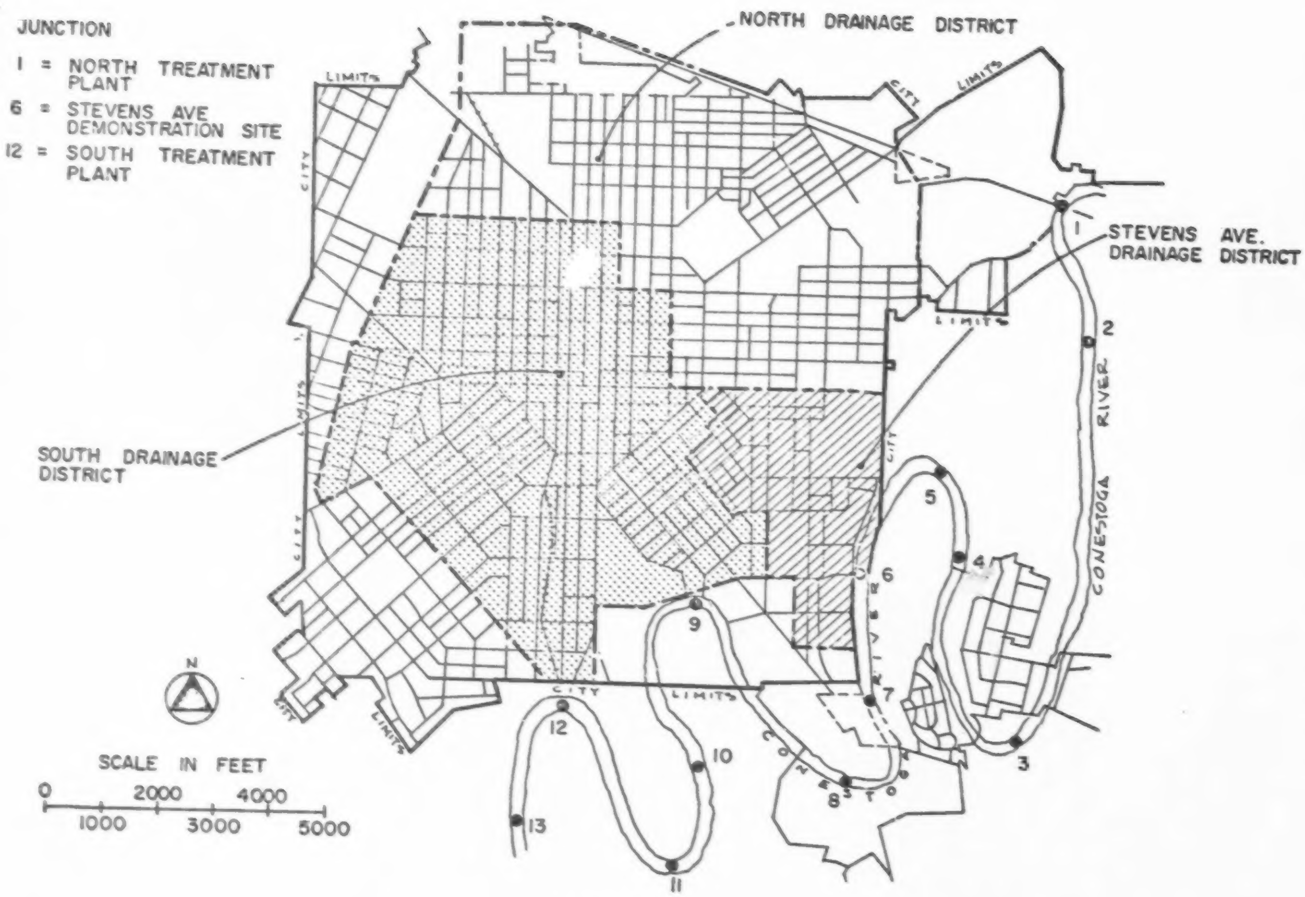

JUNCTION
1 = NORTH TREATMENT PLANT
6 = STEVENS AVE DEMONSTRATION SITE
12 = SOUTH TREATMENT PLANT
NORTH DRAINAGE DISTRICT
STEVENS AVE. DRAINAGE DISTRICT
SOUTH DRAINAGE DISTRICT
CONESTOGA RIVER
CITY LIMITS
N
SCALE IN FEET
0
1000
2000
3000
4000
5000
1
2
3
4
5
6
7
8
9
10
11
12
13

FIGURE 16.
Drainage Districts of Lancaster, Pennsylvania and Numbering System for Receiving Junctions.

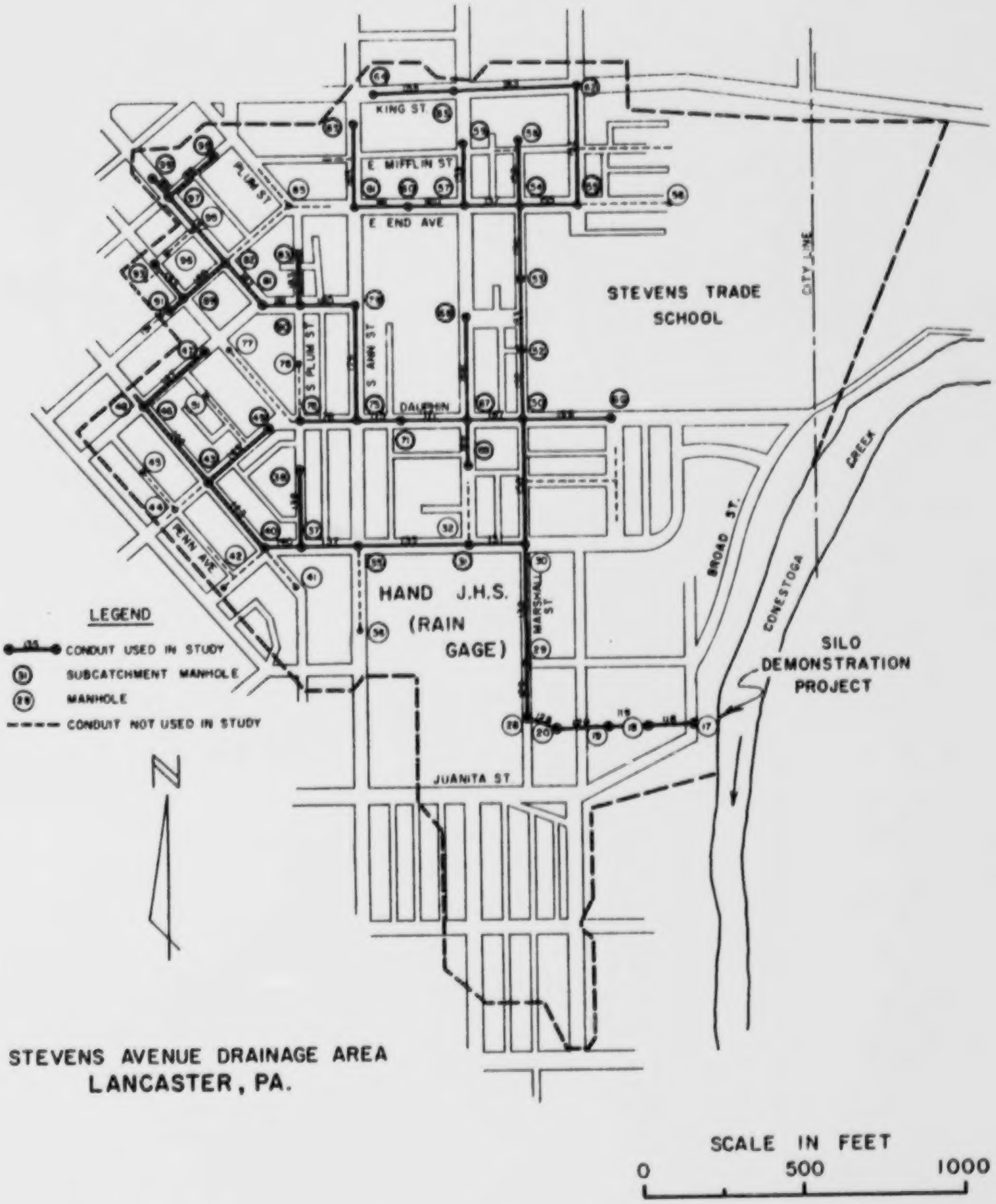
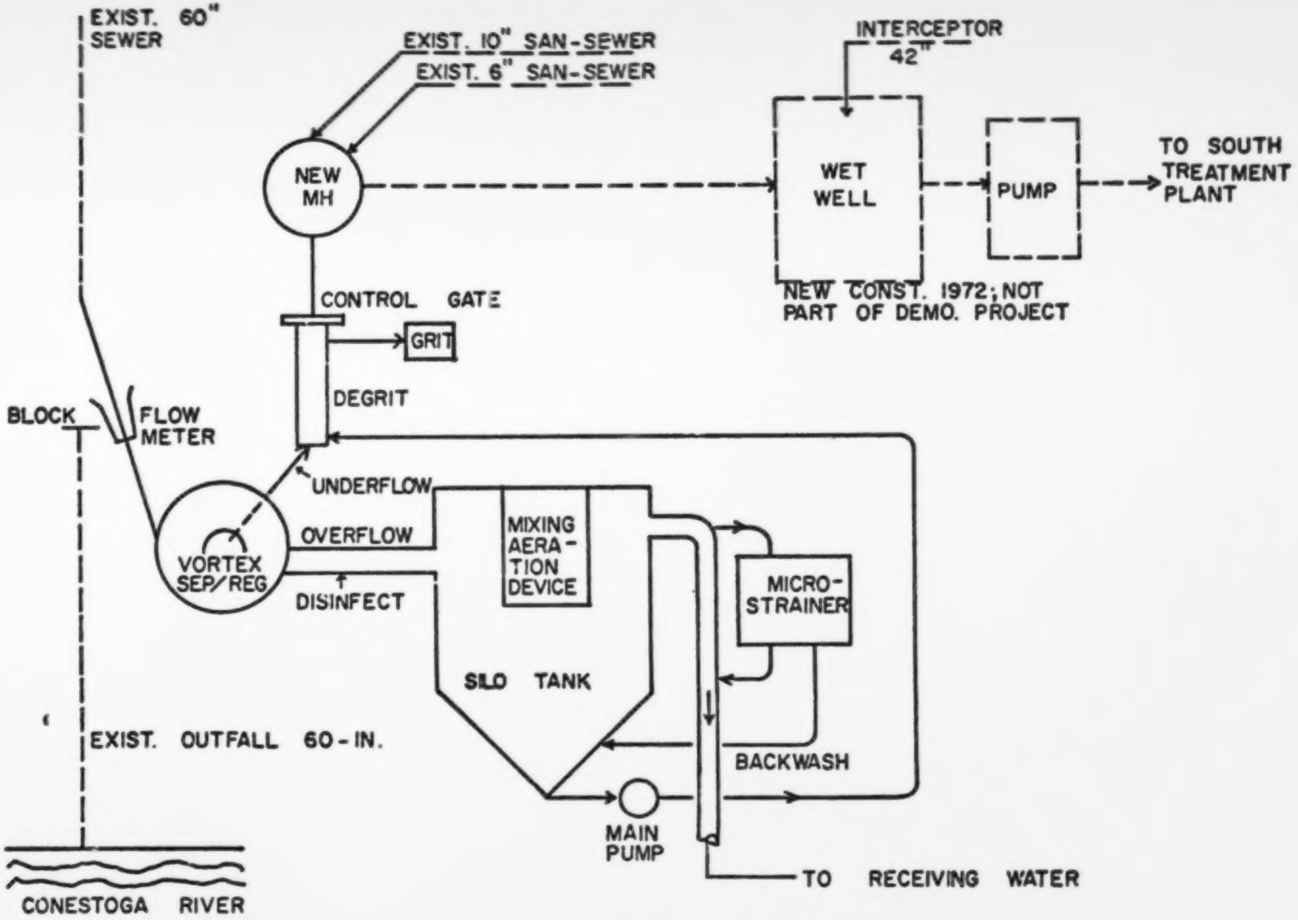

KING ST
E MIFFLIN ST
PLUM ST
E END AVE
S PLUM ST
S ANN ST
DAUPHIN
STEVENS TRADE SCHOOL
CITY LINE
HAND J.H.S. (RAIN GAGE)
MARSHALL ST
PENN AVE
BROAD ST.
CONESTOGA
CREEK
SILO DEMONSTRATION PROJECT
JUANITA ST
LEGEND
CONDUIT USED IN STUDY
SUBCATCHMENT MANHOLE
MANHOLE
CONDUIT NOT USED IN STUDY
N
STEVENS AVENUE DRAINAGE AREA
LANCASTER, PA.
SCALE IN FEET
0
500
1000

FIGURE 17. Sewer Layout of Stevens Avenue Drainage District.


EXIST. 60" SEWER
EXIST. 10" SAN-SEWER
EXIST. 6" SAN-SEWER
INTERCEPTOR 42"
NEW MH
WET WELL
PUMP
TO SOUTH TREATMENT PLANT
NEW CONST. 1972; NOT PART OF DEMO. PROJECT
CONTROL GATE
GRIT
DEGRIT
BLOCK
FLOW METER
UNDERFLOW
OVERFLOW
DISINFECT
VORTEX SEP/REG
MIXING AERA-TION DEVICE
MICRO-STRAINER
SILO TANK
BACKWASH
EXIST. OUTFALL 60-IN.
MAIN PUMP
TO RECEIVING WATER
CONESTOGA RIVER

PRELIMINARY LANCASTER FLOW DIAGRAM

FIGURE 18. Flow of Treatment-Storage Options at Demonstration Site.

C. DESCRIPTION OF THE STEVENS AVENUE RUNS

A total of three studies comprising eight storms were simulated. The City and its engineers provided input data as well as one overall set of measurements. The Stevens Avenue district was subdivided into 41 subcatchments. A description of each study and its results are given below:

Study No. 1-- The first study was based on a series of storms between July 29 and August 3, 1971. The six-day period deposited a record amount of precipitation throughout the Lancaster area (variously measured between 7.3 and 9.46 inches, or 18.54 cm and 24.03 cm respectively). During four of the six days, the storms were very intense over short periods; in one case, being the second heaviest of record. For purposes of simulation, Study No. 1 was divided into six storms. Due to the unavailability of field data for the study, verification of the results of computer simulations for each of these storms was not possible. These runs do indicate that an overflow as high as 400 cfs (11.20 m^3/sec) may be expected for a storm event similar to Storm No. 6.

Results of this study were used by APWA in sizing the swirl concentrator. A design flow of this device was established at 165 cfs (4.62 m^3/sec).

Computer simulation studies were also conducted for all six storms to evaluate the effect of the swirl concentrator-underground silo facilities on the combined overflow quality. The results of Storm No. 6 are shown on Figure 19. As illustrated, the quality of all the overflow is significantly improved through the installation of the swirl concentrator and a chlorine contact tank. Figure 19 also shows that as soon as the swirl concentrator operates at full capacity (165 cfs), excess stormwater simply overflows into the silo. The shaded portion of the flow versus time graph indicates the volume of silo storage, 160,000 ft^3 (4530 m^3). Computed flow from transport reflects the stormwater inflow to the storage facility (which is the same as the computed flow to the river without the silo simulation). The silo dewatering rate is limited by existing pumping capacity of the Stevens Avenue station (3.57 cfs or 0.10 m^3/sec). For simulation purposes, pumping was set to begin at a silo depth of 3 feet (0.91 m).

Study No. 2-- This study consisted of a storm that began in the morning of August 27, 1971 and continued almost 30 hours to the morning of the next day. It resulted in varying amounts of rainfall throughout the city averaging more than 3.5 inches (8.9 cm). The results of the computer simulation were similar to those obtained from Study No. 1, and for this reason are not included herein. Again, no measurements were taken during this study.

Study No. 3-- This study is based on a relatively minor rainfall event of March 22, 1972. This study is of special importance, however,

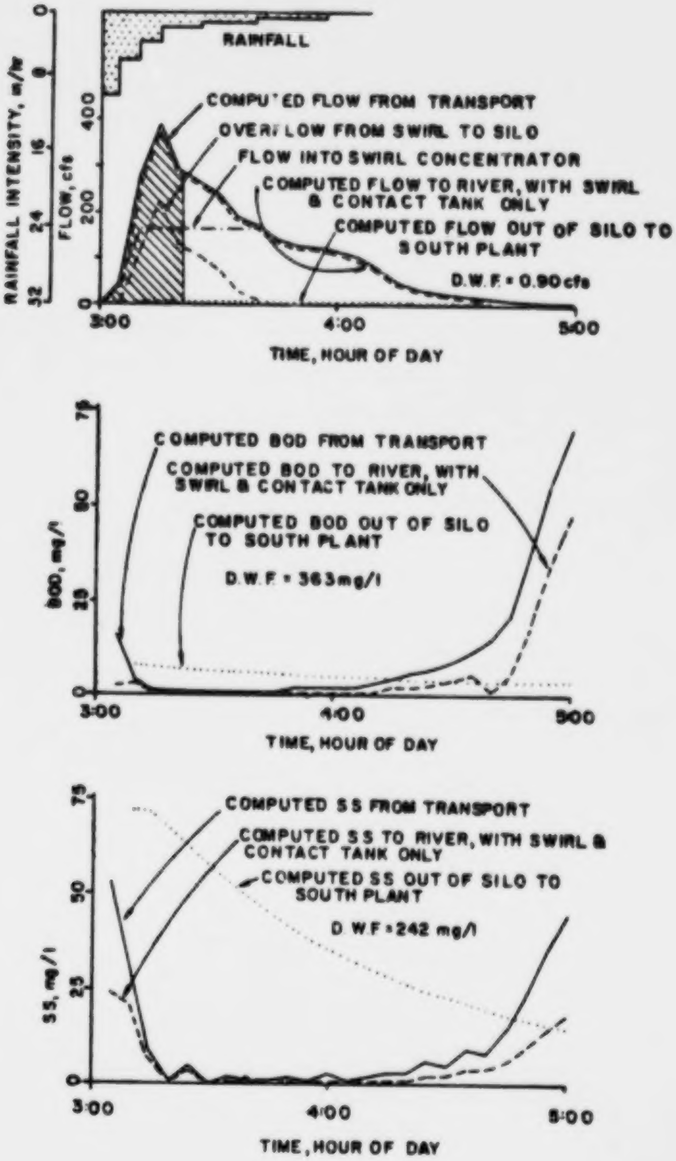


FIGURE 19. Simulation for Stevens Avenue, Study 1, Storm 6. High concentrations out of silo reflect initial flush of DWF in sewer system into the silo.

because it is one of the types most frequently experienced in terms of intensity of rainfall (return period of less than one year). It is also one for which relatively complete verification data such as rainfall, flow readings and samples were collected. The rainfall is shown in Figure 20 along with results of the computer simulation showing overflow quantity and quality. Shown in the same illustration are the actual quantity and quality measurements (points labeled x) of the overflow. It can be seen that agreement between the computer simulation and the actual measurements of flow is only fair. However, considerable doubt exists as to the validity of the flow data, since they were obtained from depths measured with a yardstick inserted into the mouth of the outfall. The agreement between the computed and measured quality parameters is only fair also. It should be noted that no calibration of the SWMM was attempted due to the lack of good verification data.

Computer simulations were also conducted on this study to determine the effect of the swirl concentrator-underground silo system. These results are also shown in Figure 20. With the silo system, the Model indicates no overflow into the Conestoga River.

D. RUNS IN THE NORTH AND SOUTH DISTRICT

Limited computer simulations were also conducted for the North and South drainage districts. The North district was subdivided into 66 catchments and the South district into 104 catchments. The sewer layouts for the North and South districts are shown in Figure 21 and 22.

Results of computer simulation for Study No. 3 for the North district are presented in Figure 23 and for the South district in Figure 24. The North district outfall is located at point 1 while the South district outfall is located at point 12 as shown in Figure 16.

An examination of these figures shows that for a rainfall event equivalent to Study No. 3, overflow from the North district would be about 100 cfs (2.8 m^3/sec) and from the South district, 160 cfs (4.48 m^3/sec). Figure 23 shows the quality concentrations of the storm water before it reaches the North treatment plant; and, subsequently, the quality concentrations of the combined sewer overflow and treatment plant effluent. Note that peaks in BOD and SS concentrations correspond to peaks in computed flow from transport. The result is that, at the two points of highest overflow, the water quality after treatment remains almost the same as that computed from transport without treatment. The dry weather flow (DWF) contribution to the Conestoga River from the South district outfall is 7.65 cfs (0.21 m^3/sec), generating approximately 10,500 lbs/day (4763 kg/day) of BOD and 7000 lbs/day (3175 kg/day) of SS. The DWF from the North district outfall is 9.50 cfs (0.27 m^3/sec), generating approximately 8500 lbs/day (3856 kg/day)

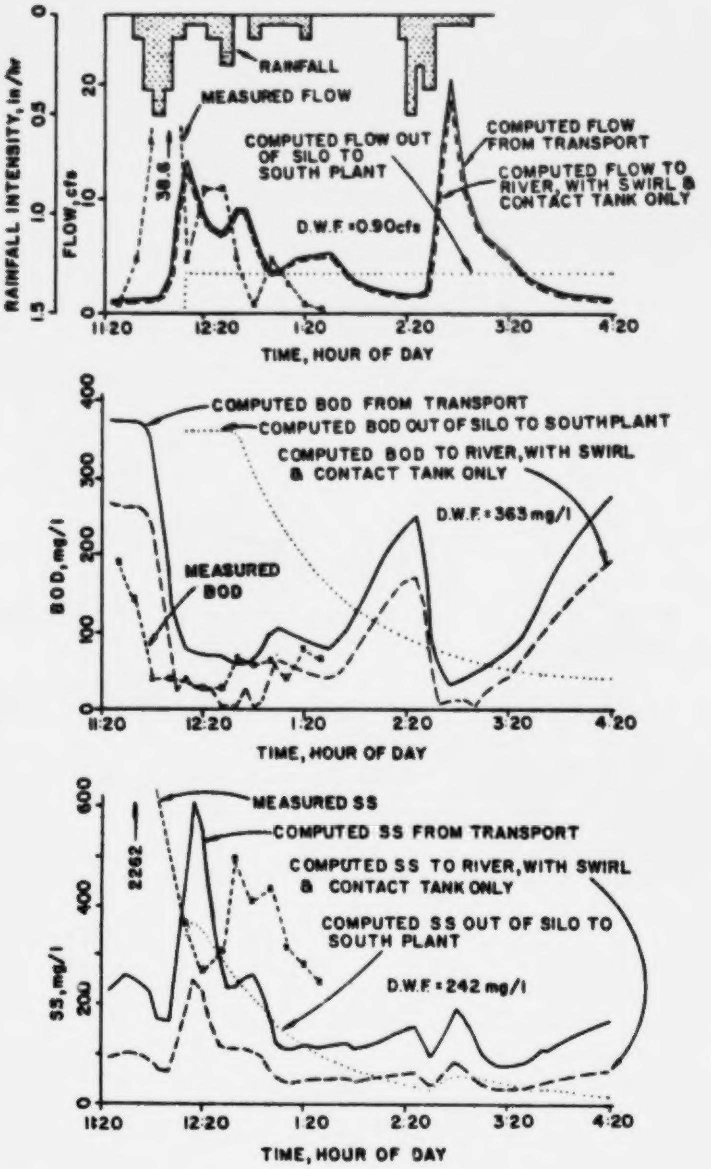


FIGURE 20. Simulation for Stevens Avenue, Study 3. High concentrations out of silo reflect initial flush of DWF in sewer system into the silo. For this storm, silo would store all runoff, but results without it are also shown.

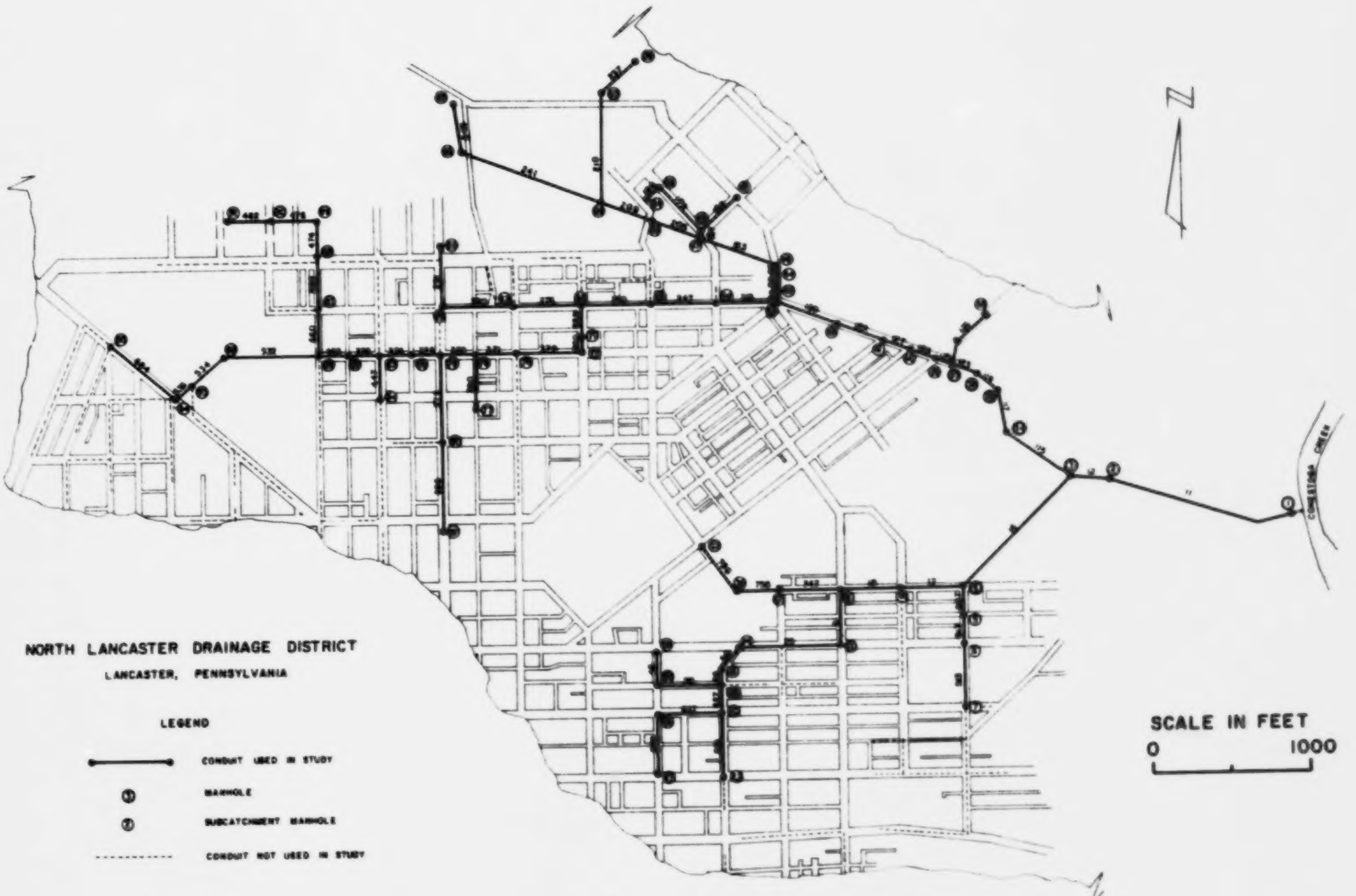

NORTH LANCASTER DRAINAGE DISTRICT
LANCASTER, PENNSYLVANIA
LEGEND
CONDUIT USED IN STUDY
MANHOLE
SUBCATCHMENT MANHOLE
CONDUIT NOT USED IN STUDY
SCALE IN FEET
0
1000
N
CONESTOGA CREEK

FIGURE 21. North Drainage District with Runoff-Transport Numbering System.

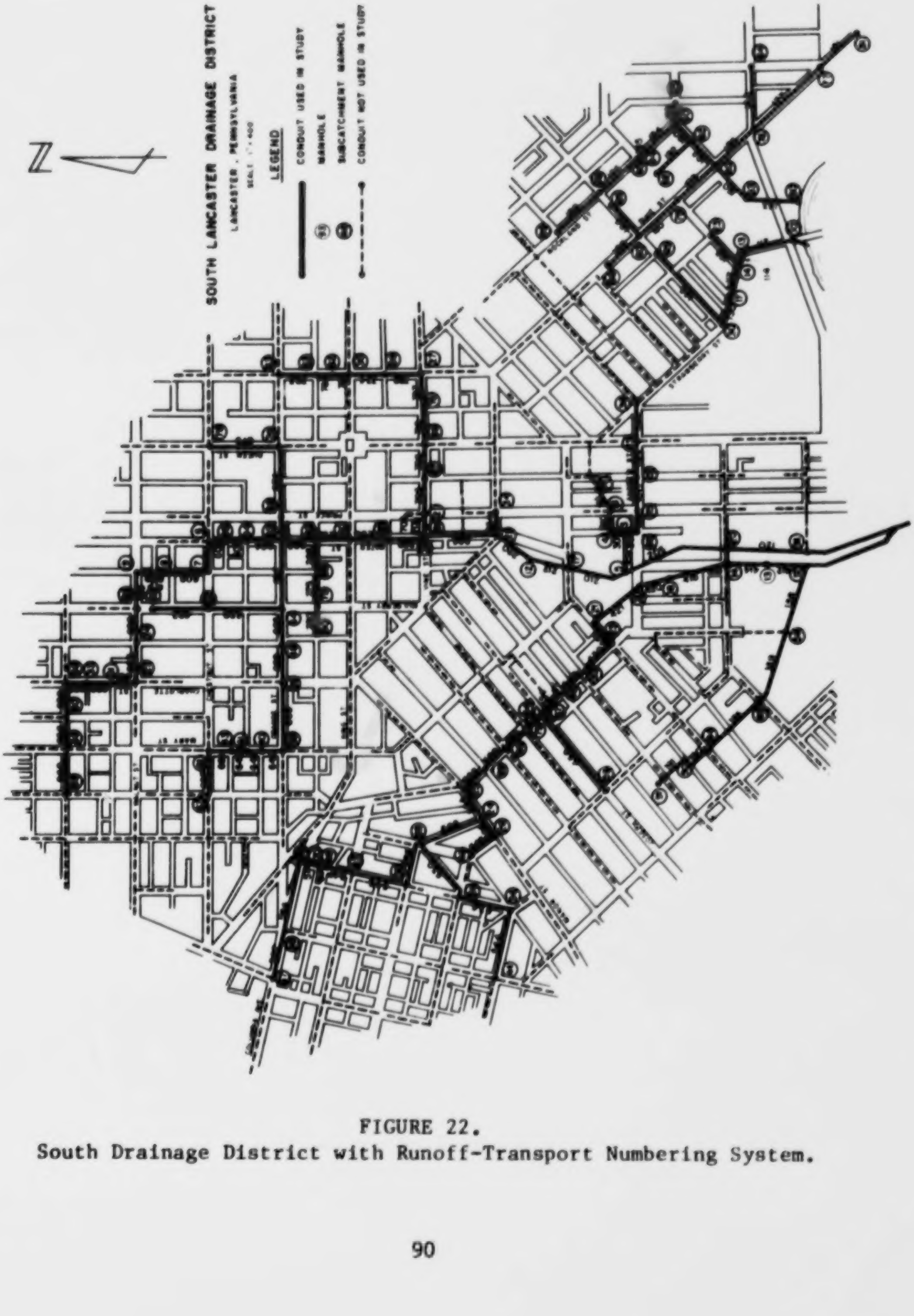

SOUTH LANCASTER DRAINAGE DISTRICT
LANCASTER, PENNSYLVANIA
SCALE 1"=400'
LEGEND
CONDUIT USED IN STUDY
MANHOLE
SUBCATCHMENT MANHOLE
CONDUIT NOT USED IN STUDY
N

FIGURE 22.
South Drainage District with Runoff-Transport Numbering System.

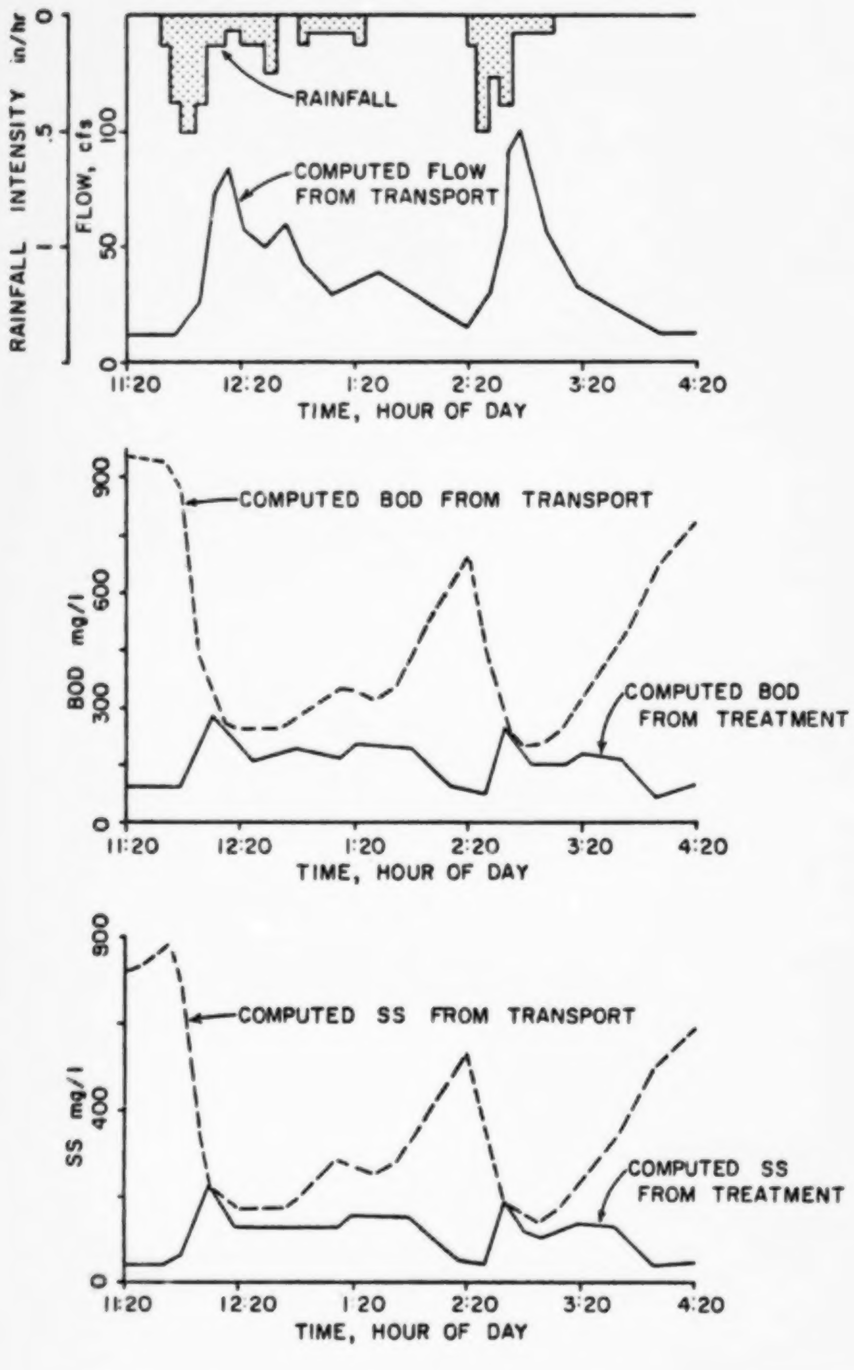


FIGURE 23. Simulation of North Drainage District, Study 3.

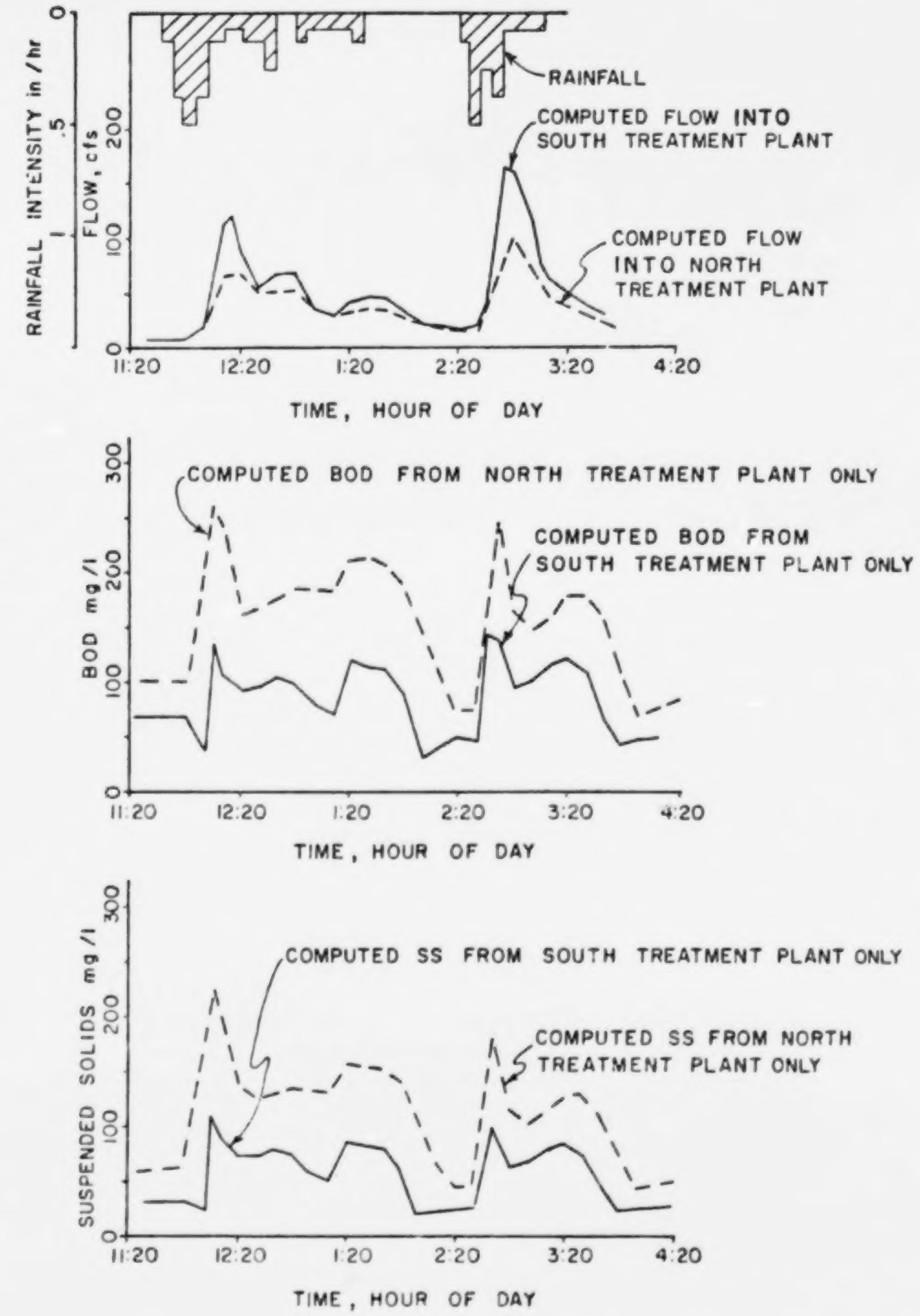


FIGURE 24. Simulation of North and South Drainage Districts, Study 3.

of BOD and 8500 lbs/day (3860 kg/day) of SS. Figure 24 compares the stormwater flow contribution and quality concentrations of the combination of treatment plant effluent and bypassed combined sewer flow for the North and South districts.

E. EFFECT ON RECEIVING WATER

To simulate the effect of the overflow on the Conestoga River, the Receiving Water Model was run on the entire city for Study No. 3. The manner in which various districts were combined is shown on Figure 25. In conducting this run, the swirl concentrator was used at Stevens Avenue while the Refined Storage and Treatment Model, as described in Section IV-D was utilized to simulate the existing biological treatment at the North and South plants. The silo was deleted in order to have an overflow at Stevens Avenue outfall since the installation of the silo prevents any overflow for a rainfall event equivalent to Study No. 3.

The reaeration coefficient for the Conestoga River was computed using the formula of O'Connor and Dobbins (1958) (Equation 14). Results of the Receiving Water Model are shown in Figures 26 through 28. Figure 26 shows DO profiles 3 and 24 hours after the storm inception. A slight DO sag is noticeable 3 hours after the storm, with minimum concentrations registered at the Stevens Avenue outfall (junction 6). The effect of the South treatment plant effluent and overflow (junction 12) is noticeable as a slight drop in DO concentration. The profiles for 48 hours and 72 hours after the storm are identical to the 24-hour profile, and were not plotted. This was true for all constituents, and it can be generally stated that the impact of the storm upon the Conestoga River is attenuated after 24 hours. Figure 27 shows the BOD profiles along the Conestoga River 3 hours and 24 hours after the storm, and Figure 28 the SS profiles. The effects of reaeration and stream self-purification are evident from examination of the pollutant input junctions (1, 6, 11) and downstream conditions. Initial values used to simulate the Receiving Water Model are listed in Table 22. The reader should note that suspended solids computations do not include a contribution from erosion sites due to a lack of data, and thus the suspended solids concentrations are somewhat low.

F. SUMMARY

Testing of the SWMM in Lancaster, Pennsylvania has revealed the importance of having sufficient and accurate measured data for model calibration. It is recommended that the SWMM user begin his sumulation from a coarse analysis of the study area. Subsequent refinement can be tailored with his knowledge of the more sensitive parameters (impervious area, detention storage, catchbasin BOD content, infiltration,

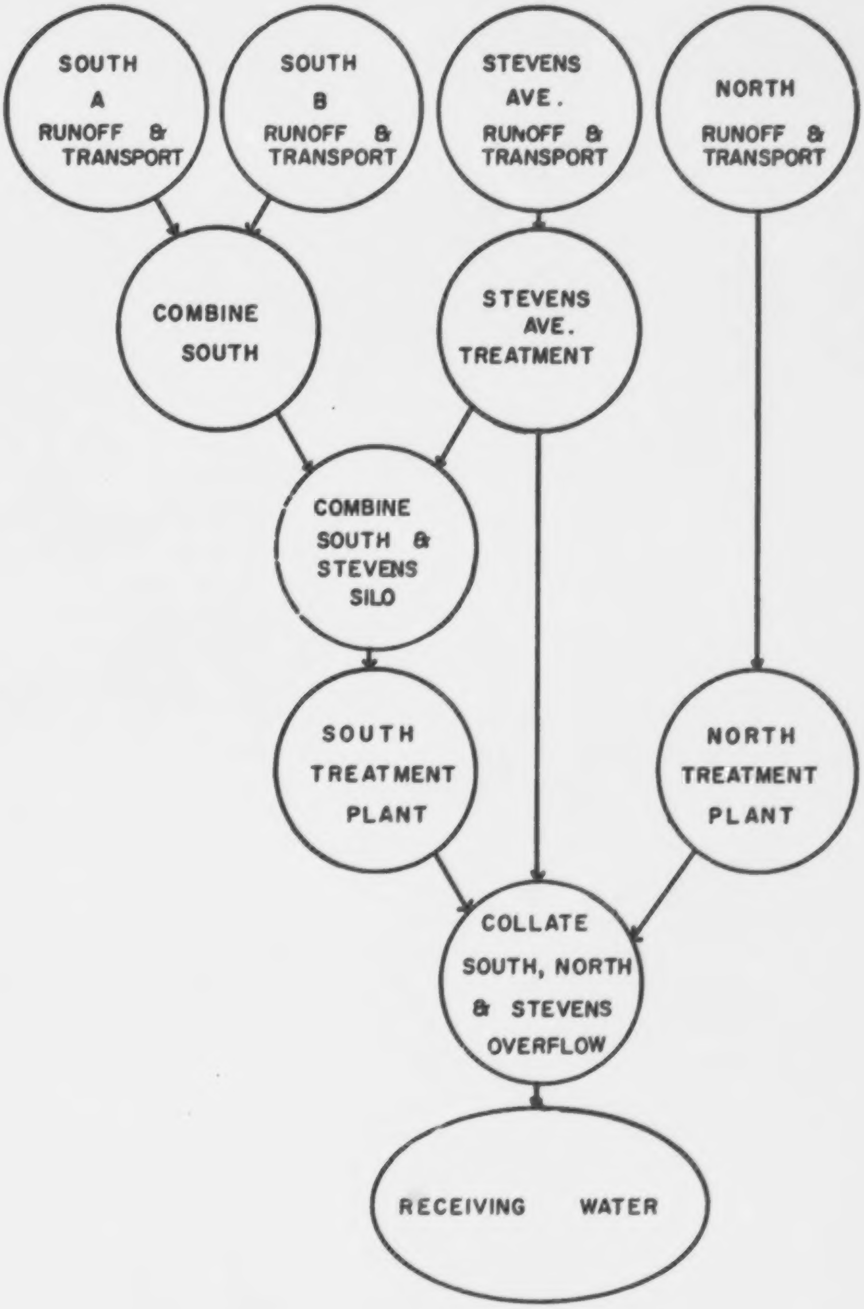

SOUTH A RUNOFF & TRANSPORT
SOUTH B RUNOFF & TRANSPORT
STEVENS AVE. RUNOFF & TRANSPORT
NORTH RUNOFF & TRANSPORT
COMBINE SOUTH
STEVENS AVE. TREATMENT
COMBINE SOUTH & STEVENS SILO
SOUTH TREATMENT PLANT
NORTH TREATMENT PLANT
COLLATE SOUTH, NORTH & STEVENS OVERFLOW
RECEIVING WATER

FIGURE 25.
Combination of SWMM Runs for Overall Lancaster Simulation.

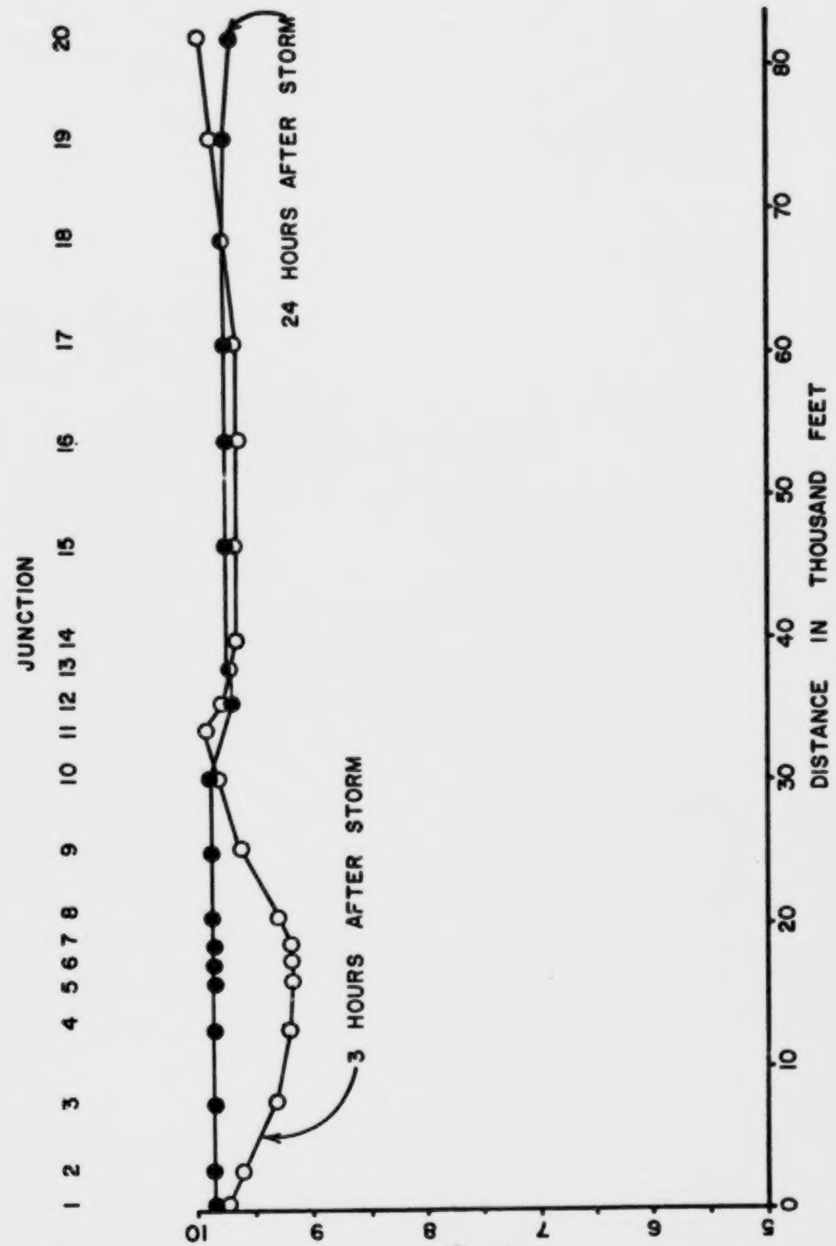


FIGURE 26.
Dissolved Oxygen Profiles Along the Conestoga River. Study 3.

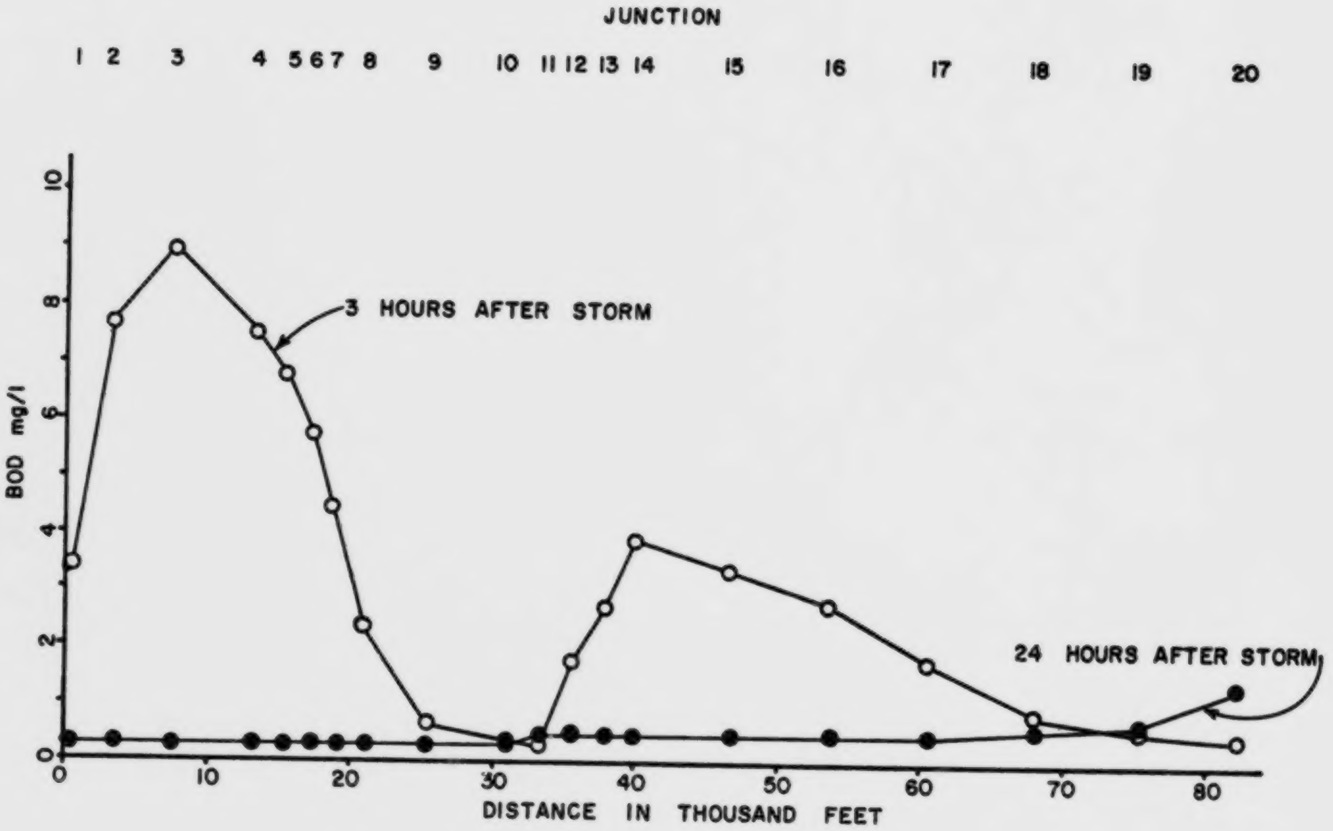


FIGURE 27.
BOD Profiles Along the Conestoga River. Study 3. During the storm reported values in the river varied between 3 and 18 mg/1.

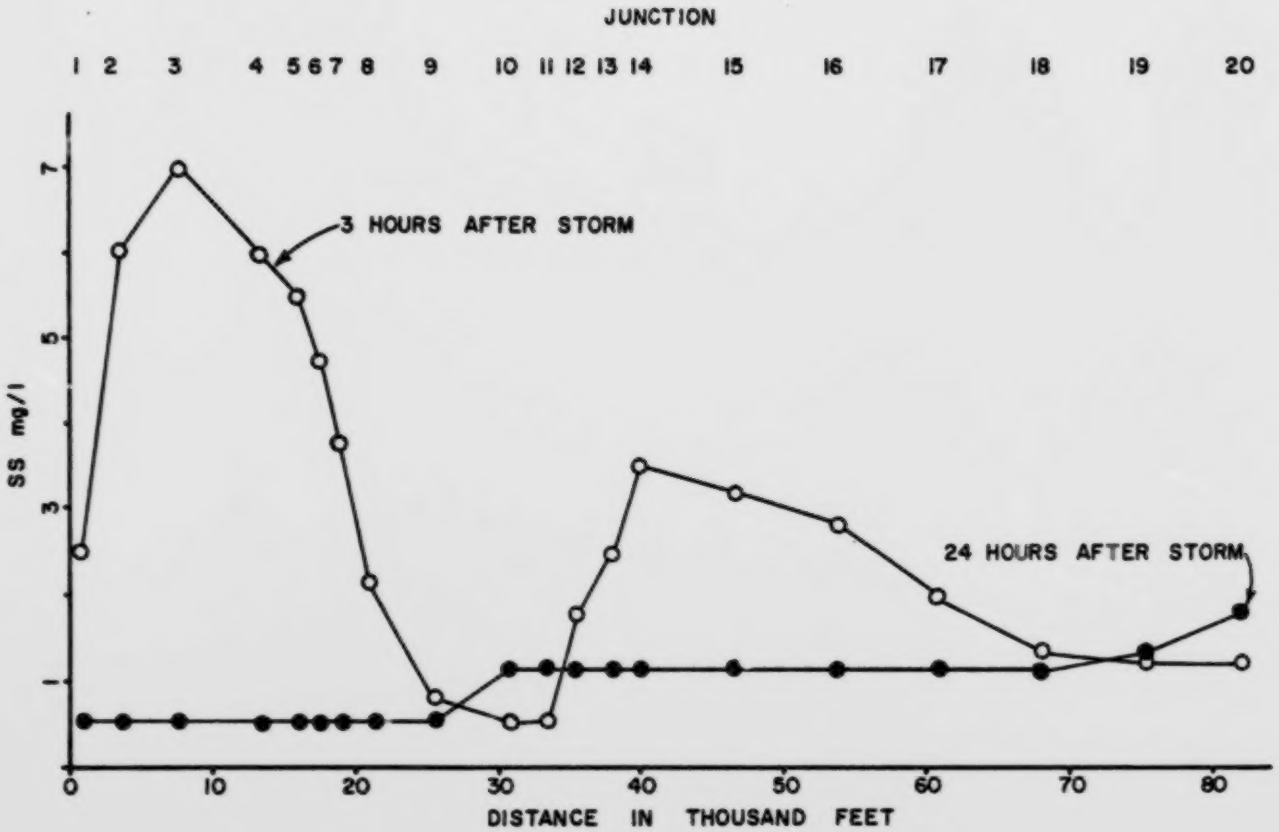


FIGURE 28.
Suspended Solids Profiles Along the Conestoga River. Study 3. Reported values in the river during the storm ranged from 35 to 155 mg/1.

Table 22. PARAMETERS USED FOR SIMULATING RECEIVING WATER MODEL

Initial and Saturation DO in Conestoga River (all junctions)	10.0 mg/l
BOD in Conestoga River (all junctions)	5.0 mg/l
Initial Suspended Solids in Conestoga River (all junctions)	10.0 mg/l
Decay Coefficient (BOD)	0.20/day
Reaeration Coefficient	1.50/day
Flow in Conestoga River (entering junction 1)	700 cfs
BOD in Conestoga River (entering junction 1)	0.0
SS in Conestoga River (entering junction 1)	0.0

and others) and available field measurements. The model is not fully calibrated until satisfactory results have been obtained with *more than one storm within the study area.* The application to Lancaster suffered from lack of good measured data for several of the storms studied, but it can be generally stated that for the Study No. 3 storm:

1) The SWMM was able to predict only fairly the quantity of the combined overflow for the Stevens Avenue district, however, the only set of measured flows are questionable.

2) Computed suspended solids were universally lower than measured values at the overflow and in the receiving water (Conestoga River) probably because erosion was not modeled and appropriate upstream concentrations were unknown.

3) Computed BOD values were higher at the overflow than the measured values, but values predicted in the Conestoga River three hours after the storm were within the range of reported values.

4) The installation of the swirl concentrator and silo complex will result in substantial improvement in the quality of the overflow at Stevens Avenue, provided the full-scale performance of the swirl concentrator is comparable to the results obtained in laboratory studies by APWA. This fact should be true for any storm comparable to those described earlier in this report.

SECTION VI

DECISION-MAKING FOR WATER QUANTITY AND QUALITY CONTROL

This section presents the results of the analysis into alternative procedures for selecting the "optimal" overall stormwater control strategy. The word "optimal" is used in a mathematical sense to denote the alternative which is best for the problem as formulated. Much use has been made of such models during the past ten years. We have been actively involved in such developments. For example, Heaney (1968) developed a mathematical programming model to allocate water supply among competing users in the Colorado River Basin. Pyatt *et al.*, (1969) and others have set up optimization models to determine the least cost combination of sewage treatment and low flow augmentation.

More recently, Kirshen, Marks, and Schaake (1972) have devised a mathematical model for analyzing stormwater quality control alternatives. A linear programming model is used to determine the sizes and operating policies for sewer pipes, storage reservoirs, and treatment plants. The SWMM is used to generate boundary conditions for the optimization model and to provide more detailed information on performance of the system.

The above optimization models assume some specified level of performance for the single-purpose system under consideration. Unfortunately, from a computational point of view, many of the stormwater quality control alternatives serve other functions such as drainage and flood control. Thus, the urban water quality problem seems to fit well into a more general framework of urban water resources management. An overall strategy for examining this more general problem has been developed. Addressing the problem in this broader perspective, the determination of the overall optimal solution can be achieved using one of several suggested procedures. A more innovative aspect of the analysis examines the questions of cost sharing among the various study areas within the urban area.

If one examines the various components of urban water resources, he is made aware of how each problem has been treated independently. Traditional concern for safe disposal of wastewater focuses on design of treatment units. There has been little systematic effort to devise overall strategies for drainage and flood control. The studies in

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Denver are a notable exception and they shall be discussed later. The question of urban water supply has been dealt with as a separate problem. Likewise, the related problems of recreation and open space have been analyzed separately from the other problems. As a result, it is difficult to gain insight as to how they all tie together.

Having analyzed these other problems, one begins to gain enthusiasm that perhaps "there is a better way." More enthusiasm was gained after participating in the layout of an 800-acre (3.24 Km^2) planned urban development which has an overall density of four units/acre and was able to satisfy the following constraints:

1) water use $\leq$ safe recharge of the aquifer;
2) on-site control of storm water from a 50-year storm;
3) no development of structures in the flood-plain;
4) no direct discharge of storm water into on-site recreational lakes; and
5) disposal of treated sewage effluent onto golf course.

This area has about 86 percent of the total land in open space. The development is for middle income groups and comprises a mix of detached dwelling units, townhouses, apartments, and condominiums. It appears that the development costs are competitive or lower than existing practices.

The following sections present the methodology used in addressing the problem. It is partitioned into the several components listed below:

A) description of the study area;
B) selection of commodities to be analyzed;
C) inventory of sources;
D) inventory of controls;
E) economic analysis of control options;

F) specification of performance criteria;

G) statement of objectives; and

H) system optimization and examination of equity question.

A. DESCRIPTION OF THE STUDY AREA

For the purposes of this analysis, assume the entire area under consideration is being partitioned into N planning units. Any subset of the N planning units will be denoted as S where $S \subset N$. In urban areas, this planning unit might be a neighborhood or a subdivision, a distinct political entity such as a ward of a city, or any other partitioning of the city which seems to be appropriate.

B. SELECTION OF COMMODITIES TO BE ANALYZED

The Storm Water Management Model (SWMM) is capable of routing water and specified conservative and nonconservative pollutants. Conceptually, it is possible to set up a decision model which simultaneously determines the optimal solution for multiple commodities flowing along the same or slightly modified network (Heaney, 1968). Such problems are classified as multi-commodity network flow problems. Unfortunately, multi-commodity models usually result in relatively large models since the number of constraints for a single commodity model is multiplied by m where m denotes the number of commodities.

It is preferable to analyze only a single commodity and incorporate consideration of other commodities implicitly. The general idea is to determine which commodity flow will be the "limiting factor" and to specify the problem accordingly. For example, a stormwater retention basin can be sized not only by hydraulic constraints but also using water quality constraints, *e.g.*, allowable suspended solids concentration in the effluent. In any given case, only one of the constraints will be binding. Thus, it is possible to incorporate water quality constraints implicitly by making appropriate adjustments in the cost functions and constraints. This is the approach we shall employ using water as the single commodity.

C. INVENTORY OF SOURCES

Early studies on the water quality aspects of the combined sewer overflows and stormwater discharges indicate that this water may have a significant pollution load. Often a "first flush" phenomenon was observed, *i.e.* a high initial pollutant concentration and subsequent rapid decrease in concentration. Usually it was assumed that the pollution emanated from material that had settled out in the sewers during antecedent dry weather, from catchbasin sumps, from street gutters, and/or from rooftops, parking lots, and related areas.

Studies in Chicago provided an estimate of the type of materials found in urban areas (Heaney and Sullivan, 1971). The results are shown in Table 23. The identification of the potential magnitude of the stormwater quality problem inspired subsequent efforts to determine the origin of these pollutants. A summary of these efforts is presented below.

1. *Air Pollution*

Air pollutants from point and non-point sources are a significant component of the stormwater quality problem. Precipitation in urban areas acts as a "scrubbing" device to cleanse the atmosphere and to flush away air pollutants which have settled in the urban area. From an environmental control point of view, it is important to understand the relative merits of source control of air pollution *vs.* alternative control of stormwater pollution.

Fortunately, parallel work has been done in developing air quality management models. The model which is frequently used for this purpose is called the Air Quality Display Model (AQDM). Like the SWMM it is capable of estimating the transport of pollutants (particulates and SO_2), the cost of control, and the impact of alternative control strategies. This model is available from EPA (TRW, 1969). The University of Florida has utilized this model in studies of air pollution control in Jacksonville, Florida, and found it to be a useful tool (Wilson, 1971). Thus, air pollution levels can be estimated using such approaches. Air pollution from specific industries can be estimated using information from EPA sponsored studies. For example, Heaney and Doughty (1971) developed a model for determining the optimal control strategy for emissions from any type of paper and pulp mill.

The above air pollution models have not been incorporated directly into the SWMM since this analysis can be easily done as separate special studies of sources of stormwater pollution.

Table 23. ESTIMATED MONTHLY QUANTITY OF IDENTIFIABLE SOLIDS FROM 10-ACRE URBAN AREA, CHICAGO, ILLINOIS
(Quantity of Solids Sources: tons/month)
(Heaney and Sullivan, 1971)

	Exterior Solids						
			Refuse				
Month	Air pollution	Domestic wastes	Garbage	Other	Street sweepings	Catch basins	Totals[a] tons/mo.
January	0.68	2.4	1.8	7.2	0.68	.43	13.2
February	0.78	2.4	1.8	6.8	0.68	.43	12.9
March	0.80	2.4	2.3	7.9	0.76	.43	14.6
April	0.60	2.4	2.3	8.8	0.76	.43	15.3
May	0.72	2.4	2.3	8.5	0.76	.43	15.1
June	0.40	2.4	2.8	9.2	0.76	.43	16.0
July	0.46	2.4	2.8	8.7	0.76	.43	15.6
August	0.30	2.4	2.8	9.3	0.76	.43	16.0
September	0.45	2.4	2.3	8.9	0.76	.43	15.3
October	0.56	2.4	2.3	8.3	1.51	.43	15.5
November	0.61	2.4	2.3	8.4	1.51	.43	15.6
December	0.66	2.4	1.8	8.2	0.68	.43	14.2
TOTAL[a] tons/yr.	7.00	28.8	27.6	100.2	10.40	5.20	179.3

[a] Totals are rounded off

2. *Sedimentation and Erosion*

It became evident early in the analysis that it was essential to better characterize the nature of the settleable and suspended solids in stormwater runoff. That portion of the solids load attributable to air pollution can be estimated using the AQDM. Field observations in Lancaster, Pennsylvania; Gainesville, Florida, and elsewhere indicate that the predominant materials in the residential areas result from localized soil erosion due to construction, parking, etc. Thus it is possible to characterize the nature of the solids as a function of the soils within the study area. This information is extremely helpful in designing control facilities such as settling basins.

Thus, a method for estimating erosion has been included in the SWMM. This approach uses the "Universal Soil Loss Equation" which was originally developed for agricultural areas. A detailed description of this phase of the investigation is presented in Section IV-B.

3. *Yard Litter*

In addition to erosion, yard litter appears to comprise an important fraction of the potential pollutants in stormwater. The quantity of vegetation entering the sewer system depends primarily on local conditions and practices. For example, yard waste in Gainesville, Florida is piled in the street gutter until the next collection. If a storm occurs during the interim, then some of this material may be flushed into the sewers. Thus, a separate study was made of this problem. The results, presented in Section IV-B, indicate the importance of this component as a major source of organic material.

4. *Other Sources*

In addition to the above specific studies, other efforts have examined the specific impact of highway de-icing chemicals (EPA, 1971) while numerous efforts have been made to characterize the general magnitude of the problem (*e.g.*, AVCO, 1970). As these studies are completed, our understanding of the origin of this material will improve accordingly.

5. *Quantity of Urban Stormwater*

From a hydrologic point of view, urbanization with storm sewers causes significant increases of peak flows in the receiving waters. This is a prime concern from the point of view of downstream flooding, instream erosion, and undesirable effects on the receiving water itself.

These effects are becoming more important as the perceived function of these receiving waters changes from their traditional use as drainage canals to a growing awareness of their value as an environmental asset.

In general, urbanization causes a shift in the original hydrograph as shown in Figure 29. Estimates of the increase in peak discharge due to urbanization are shown in Figure 30.

Engineering design is usually based on providing a facility which can handle some "worse case" condition. In this case, one might attempt to size the control facilities to handle the peak flow. However, this approach is quite expensive since the peak is so high and is of very short duration. Because of the nature of the problem, it is logical to place heavy emphasis on incorporating some kind of storage facilities into the design.

Storm sewer designers typically use the Rational Method to estimate the peak rate of flow. More sophisticated methods which incorporate routing **flow** procedures are also available. One might alternatively or conjunctively try to estimate the total *volume* of runoff from a given storm. As development increases, the volume of runoff increases. Thus, the given storm produces not only a higher peak rate of runoff, but also a larger total volume of runoff due to the reduced opportunity for soil infiltration.

Urban hydrologists have devised techniques for estimating the total volume of runoff which are quite useful (see Chapter 20 of Chow, 1964, for example). The Soil Conservation Service (1972) has developed very comprehensive procedures for analyzing the hydrology of agricultural areas. Their procedure for estimating runoff is especially useful for examining urbanizing watersheds. Also related manuals have been developed by SCS for design of control facilities such as grassed waterways, storage ponds, etc. This work has proven invaluable in determining the magnitude of the sources of stormwater.

This section has attempted to characterize the sources of stormwater pollution and to briefly describe the general impact of urbanization on the rainfall-runoff relationship. With regard to the hydrologic aspect, one can see that an ever-increasing rate of runoff and volume of runoff accompany urbanization. Thus, controls need to be designed for more stringent storage and treatment criteria. Studies of the origin of pollutants in stormwater indicate that the nature of the pollutants could be expected to change as an area undergoes urbanization. In urban areas with a relatively large amount of pervious area, one would expect that soil erosion and yard litter would comprise the

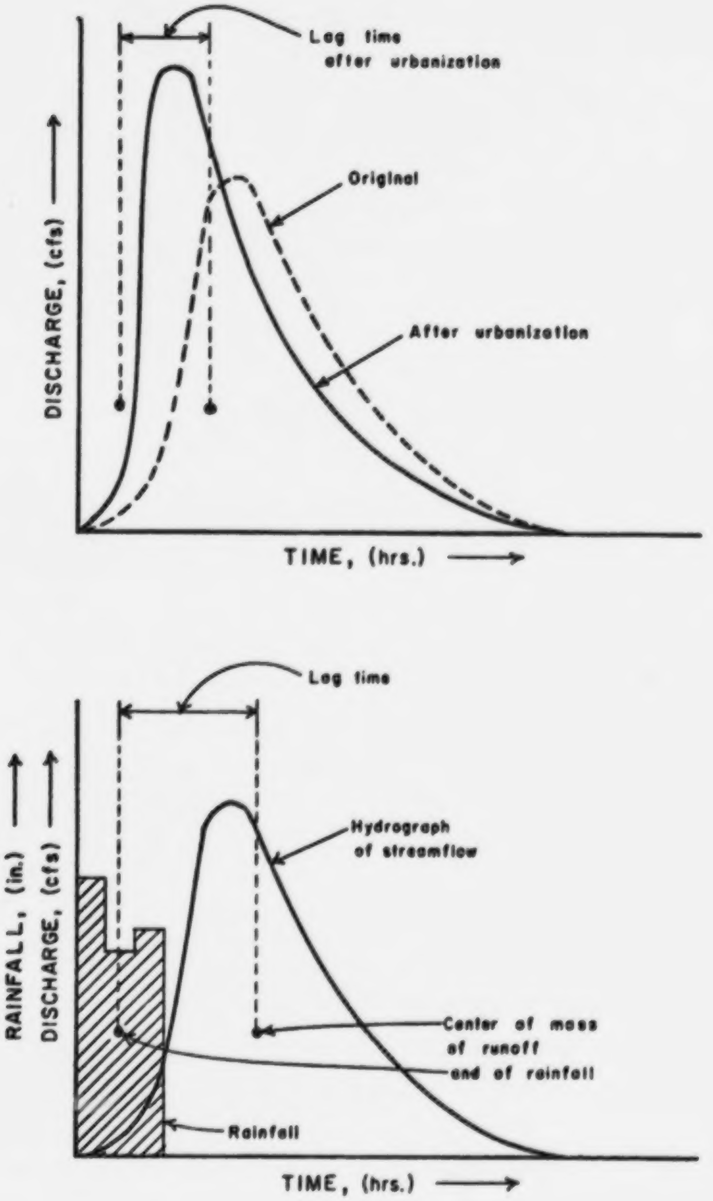


FIGURE 29.
Effect of Urbanization on Storm Water Hydrograph
(Leopold, 1968)

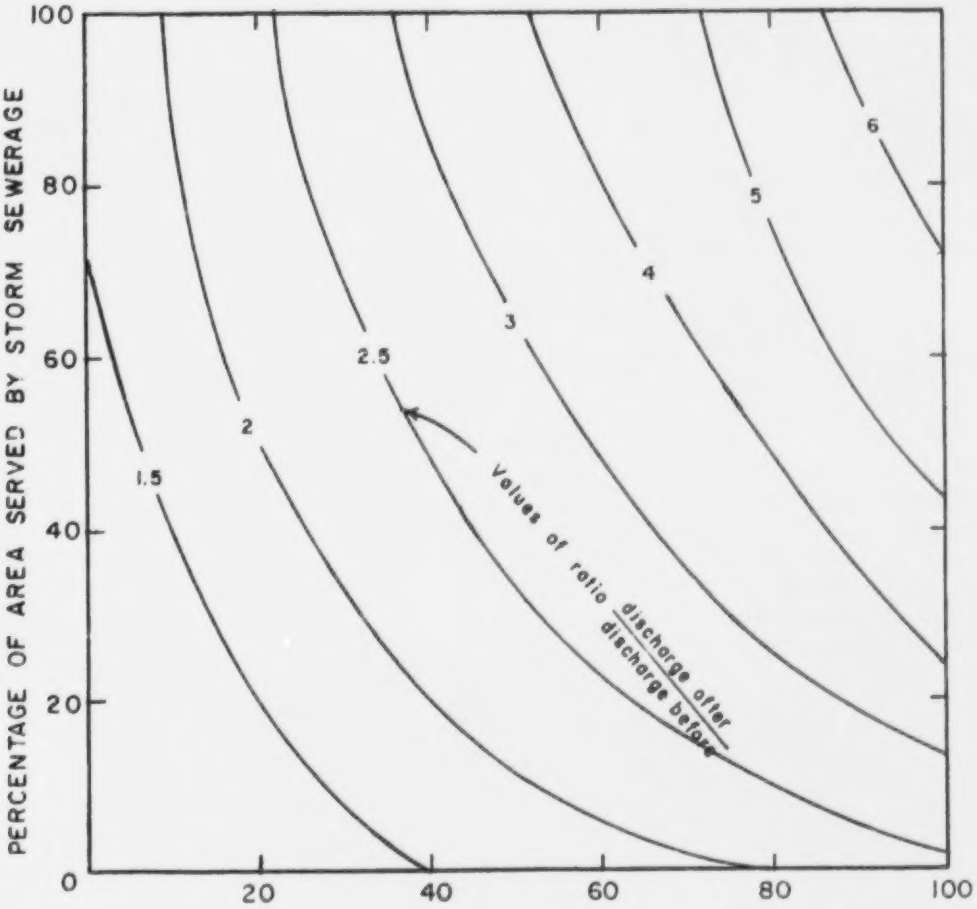


FIGURE 30. Effect of Urbanization on Peak Discharges (Leopold, 1968)

most important sources of pollution. Correspondingly, as the urban area is rendered impervious by further development, these sources would be reduced and pollution emanating more from de-icing chemicals, air pollution, grease, oil, etc., which typify highly urbanized areas, would increase in relative importance.

D. INVENTORY OF CONTROLS

1. *Water Quality Controls*

It is evident from the inventory of sources that a wide variety of controls might be employed to reduce problems associated with specific pollutants and increased flows. An excellent summary of alternative water quality control devices has been presented by Field and Struzeski (1972). A summary of the water quality controls which they discuss is presented in Table 24. Many of these control options have been incorporated directly into the SWMM.

2. *Water Quantity Controls*

A thorough review has been made of urban storm drainage practices in order to better understand the rationale behind present system design. Storm sewer design is a standard procedure which engineers have routinely performed for a number of years. Referring to Figure 30, one can see that the increase in peak flow is a function not only of the percent impervious area but also the percent of the area served by storm sewers. There is not *necessarily* a one-to-one correspondence. As can be seen from Figure 30, it is possible to significantly reduce the peak flows if one could discourage the use of storm sewers and/or encourage the discharge onto pervious areas.

Subsequent subsections will examine current design criteria for streets, parking, and roofs in urban areas. The purpose of these sections is to examine the feasibility of modifying their design to help ameliorate urban stormwater problems.

According to an unpublished Rand study of land use in 48 U.S. cities, the land use proportions are as follows (Urban Land Institute, 1968):

Type of Use	*% of Total Land Use*
Residential	39.0
Industrial	10.9

Table 24. CONTROL OF COMBINED SEWER OVERFLOWS (Field and Struzeski, 1972)

A. *Existing System Control*

1. Maximizing treatment
2. Improved regulator maintenance
3. Infiltration and extraneous inflow control
4. Surface and sewer system "housekeeping"
5. Polymers to increase flow capacity

B. *Advanced Control Systems*

1. Flow regulation
2. Storage
3. Porous pavement
4. New sewer systems

C. *Treatment Methods*

1. Fine-mesh screening and microscreening
2. Dissolved-air flotation
3. Rotating biological contactors
4. High-rate plastic media trickling filters
5. High-rate, single-, and dual-media filtration
6. Vortex, swirl, and helical separators
7. Advanced disinfection methods, high-rate application, on-site generation, automated operation, ozonation, and use of combined halogens (chlorine and iodine) and chlorine dioxide
8. Tube settlers
9. Powdered and granular activated carbon adsorption
10. Polymer and other chemical additives for improved settling, microscreening, filtration, and flotation
11. Chemical oxidation
12. In-line or in-sewer treatment
13. Sludge handling and treatment
14. Regeneration of carbon and coagulants
15. Reclamation and reuse

Commercial	4.8
Road and Highway	25.7
Other Public	19.6
	100.0

For Lancaster, Pennsylvania, the study area for this report, the impact of 1000 new inhabitants has been estimated (Urban Land Institute, 1968). The 1000 inhabitants would comprise about 270 families with 500 adults, 200 pre-school children, and 300 school children. The land requirements for these new residents were estimated as shown below:

Type of Use	*Acreage*
Residential	90
Street	13
Public Land	20
Service	3
Retail Stores	2
	128

Both of the above tables indicate that residential areas and streets are the predominant types of land use and consequently have the greatest impact on urban stormwater problems. Some of the newer concepts in urban planning such as planned unit development (PUD) and density zoning control offer hope of improved storm drainage practice. For example, a new PUD north of Tampa, Florida handles all stormwater on-site and maintains over 80 percent of the land in open space. Comparisons of density zoning control with standard subdivision layouts are shown in the *Community Builders Handbook* (Urban Land Institute, 1968). These newer land use forms appear to offer the long-run solution to current stormwater quality management problems.

The following subsections present a summary of current practices in design of streets and parking areas. Also, a brief description of rooftops is included. These sections provide a first glance at the feasibility of stormwater control by modifying current design practices.

Parking

The research into parking has provided an interesting example of how a major urban land use category has evolved with little consideration for its overall impact. A recent publication by the Highway Research Board (HRB) (1971), titled *Parking Principles*, provides a good summary of the state of the art. One frequently suggested method for controlling stormwater problems is to temporarily pond water on parking lots and perhaps on streets. This section discusses the general present criteria underlying design of off-street and on-street parking facilities.

A primary recommendation of the HRB report is that on-street parking be discouraged because of the traffic hazard. Certainly there has been a strong movement during the past 20 years towards that goal.

In residential areas, on-street parking is permitted with the following constraints according to the 1968 edition of the Uniform Vehicle Code as reported in the HRB report. Parking shall be prohibited in the following locations:

1) on a sidewalk;

2) in front of a driveway;

3) within an intersection;

4) within 15 feet (4.5m) of a fire hydrant;

5) on a crosswalk;

6) within 20 feet (6.1m) of a crosswalk at an intersection;

7) within 30 feet (9.1m) on the approach to any flashing beacon, stop sign, or traffic-control signal located at the side of a roadway;

8) between a safety zone and the adjacent curb or within 30 feet (9.1m) of points on the curb immediately opposite the ends of a safety zone, unless the traffic authority indicates a different length by signs or markings;

9) within 50 feet (15.2m) of the nearest rail of a railroad crossing;

10) within 20 feet (6.1m) of the driveway entrance to any fire station within 75 feet (23m) of said entrance (when properly posted);

11) alongside or opposite any street excavation or obstruction when stopping, standing, or parking would obstruct traffic;

12) on the roadway side of any vehicle stopped or parked at the edge or curb of a street; and

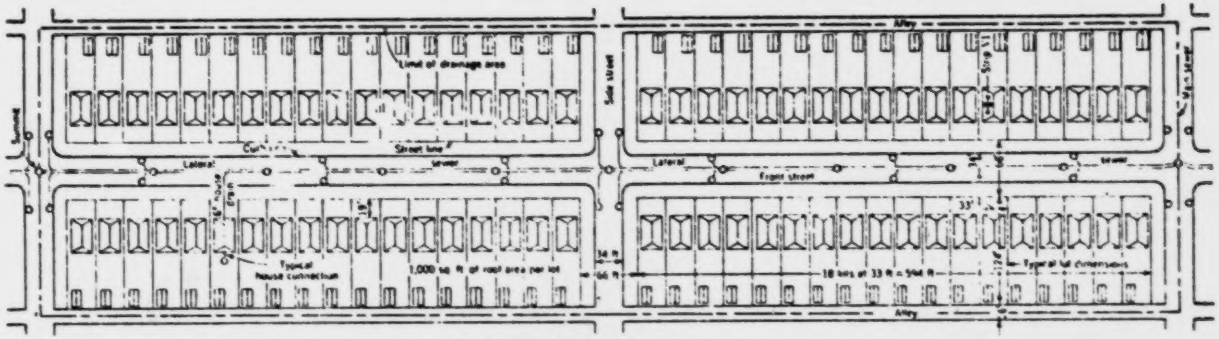
13) on any bridge or other elevated structure on a highway or within a highway tunnel.

Let us examine street layout such as exists in Chicago wherein a typical block is 660 feet x 330 feet (201 m x 101 m). A section of this street is shown in Figure 31. Note that the section does not have driveways and cars may be parked either on the street or in the garage which is accessible via the alley.

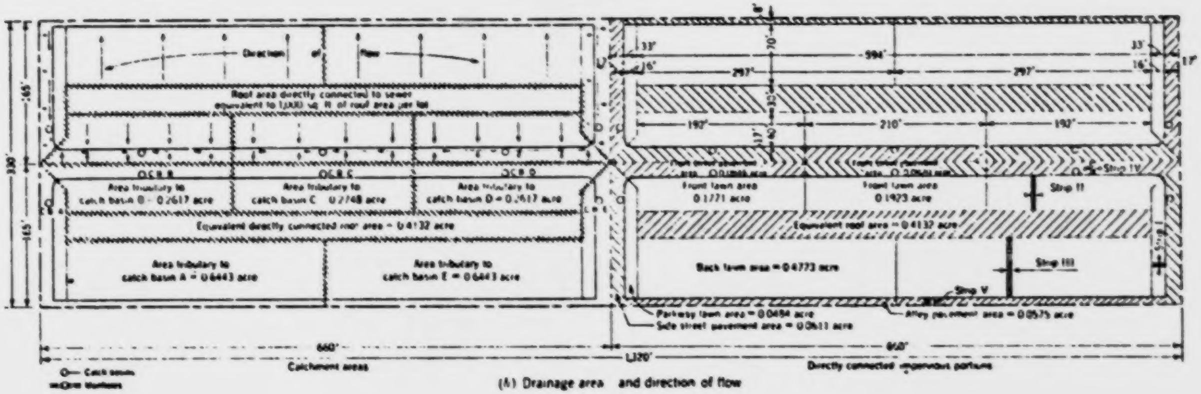
For the land use shown in Figure 31, there is about 1400 feet (430 m) of parking available on the street. The required length of parking space is about 22 feet (6.7 m) for a full-size car. Thus, there are about 64 spaces.

This section of Chicago is typical of residential land use in many older cities in the United States. This land use pattern can be compared with lower densities by reducing its density. The results of this exercise are shown in Table 25. As the density is reduced, it is assumed that alleys are replaced by driveways and the roof area/house increases. The percent imperviousness of the original section is 36 percent. It is interesting to note that it is still 22 percent at a low density of 1.2 dwelling units/acre (0.5 units/ha).

The last row of the table illustrates vividly the increase in paved area/dwelling unit as density increases. This reflects a significant reduction in the utilization of available parking and traffic lanes. As urban residents make the transition to using driveways, two related factors reduce the effective utilization of paved area: 1) the increased paved area for the driveway; and 2) the reduction in on-street parking spaces due to restrictions on parking in front of or near driveways. Thus, changing land use patterns have increased significantly the paved area/dwelling unit and reduced the effective utilization of the paved area.


Alley
Summit
Limit of drainage area
Side street
Strip VI
Curb
Street line
Lateral
sewer
Front street
6" house drain
19'
Typical house connection
1,000 sq ft of roof area per lot
34 ft
66 ft
18 lots at 33 ft = 594 ft
33'
Typical lot dimensions
Alley

(a) Land use and sewerage


Direction of flow
Roof area directly connected to sewer equivalent to 1,000 sq ft of roof area per lot
OCB B
OCB C
OCB D
Area tributary to catch basin B = 0.2617 acre
Area tributary to catch basin C = 0.2748 acre
Area tributary to catch basin D = 0.2617 acre
Equivalent directly connected roof area = 0.4132 acre
Area tributary to catch basin A = 0.6443 acre
Area tributary to catch basin E = 0.6443 acre
165'
165'
330'
660'
Catchment areas
1,320'
O— Catch basin
33'
16'
594'
297'
297'
70'
30'
192'
210'
192'
17'
Front lawn area 0.1771 acre
Front lawn area 0.1923 acre
Strip II
Strip III
Strip IV
Strip V
Equivalent roof area = 0.4132 acre
Back lawn area = 0.4773 acre
Parkway lawn area = 0.0484 acre
Side street pavement area = 0.0611 acre
Alley pavement area = 0.0575 acre
660'
Directly connected impervious portions

(b) Drainage area and direction of flow

FIGURE 31.
Typical Chicago 10-acre Tract Drainage Basin
(ASCE, 1960)

Reproduced from best available copy.

Table 25. EFFECT OF HOUSING DENSITY ON LAND USE PATTERNS

Item	Housing density in units per acre				
1) Land use	14.40	7.20	3.60	2.00	1.20
a) Total paved area	.97	.97	.99	.88	.82
i) Front street	.50	.50	.50	.50	.50
ii) Side street	.24	.24	.24	.24	.24
iii) Alley	.23	.23	.00	.00	.00
iv) Driveway	.00	.00	.25	.14	.08
b) Total roof area	.83	.83	.62	.46	.34
c) Pervious area	3.20	3.20	3.39	3.66	3.84
i) Front and rear lawn	3.00	3.00	3.19	3.46	3.64
ii) Side lawn	.20	.20	.20	.20	.20
d) Total of a, b and c	5.00	5.00	5.00	5.00	5.00
2) Per cent impervious	36	36	32	27	23
3) Per cent to sewer system	36	36	15-32	15-27	15-23
4) Dwelling units	72	36	18	10	6
5) Typical lot size	(33x124)	(33x124)	(66x132)	(119x132)	(198x132)
6) Impervious area/ dwelling unit	.025	.05	.089	.134	.227

Off-street parking has been encouraged not only for residential areas but also for all other types of land use. The HRB report recommends the number of parking spaces for various types of land use (see Table 26). For example, parking for shopping centers is estimated to require 5.5 spaces/1000 square feet (59 spaces/1000 m^2) of gross leased area. For a one-story center, this amounts to about 2 square feet of parking area/square foot of roof area. According to the *Community Builders Handbook* (1968), this design criteria accommodates the need for parking spaces for all but the 10 highest hours of demand during the entire year. These peak hours occur during three days of the year. Including cross aisles and entrances, about 350 square feet (33 m) of paved area/parking space is needed to accommodate full-sized American cars. Smaller spaces are required for the more compact vehicles. Table 27 shows recommended stall dimensions for various cars.

It is apparent from Table 27 that full-size American cars require an additional 50 percent paved parking area/vehicle. Parking facilities are usually designed for this worst case condition. A summary of the dimensions of 1973 cars produced by Ford Motor Company provides a comparison of vehicle dimensions.

Table 28 shows that the larger American cars would require a much larger parking area and a better paving material to handle their heavier weight (about twice the weight of a sub-compact).

The above analysis of parking in urban area illustrates the impact of the automobile on urban land use. The desire for off-street parking and larger vehicles has necessitated a significant increase in per capita parking demand.

From the water quality control point of view, each square foot of paved area increases stormwater problems so that it would be helpful to discourage unnecessary construction of such facilities. It is also desirable to encourage temporary on-site detention of stormwater in parking lots. If one surveys parking facilities in newer urban areas, he observes a relatively low overall rate of utilization. For example, a church might have off-street parking which is actually used only a few hours a week. It is considered good practice for a commercial establishment to provide enough spaces to accommodate the peak load of cars.

Residential areas with driveways require a significant increase in paved areas which is not utilized too often. A value judgment is required to evaluate whether such practices are wasteful.

Table 26. ZONING STANDARD GUIDELINES FOR PARKING
(Highway Research Board, 1971)

Use of building site	Minimum number of parking spaces required
Residential	
Single family	2.0 per dwelling unit
Multifamily[a]	
Efficiency	1.0 per dwelling unit
One or two bedrooms	1.5 per dwelling unit
Three or more bedrooms	2.0 per dwelling unit
Commercial	
Offices[a] and banks	3.3 per 1,000 sq ft GFA[b]
General retail[a]	4.0 per 1,000 sq ft GFA[b]
Shopping centers	4.4 per 1,000 sq ft GLA[c]
Restaurants[a]	0.3 per seat
Hotels, motels[a]	1.0 per rentable room plus 0.5 per employee
Industrial[a]	0.6 per employee
Auditoriums and theaters[a]	0.3 per seat
Churches	0.3 per seat
College/university	
Good transit access	0.2 per student
Auto access only	0.5 per student
Senior high school	0.2 per student plus 1.0 per staff member
Elementary and junior high school	1.0 per classroom
Hospitals	1.2 per bed

[a] Exceptions permitted in Central Business District if adequate public transportation is available.

[b] GFA = Gross Floor Area

[c] GLA = Gross Leased Area

Table 27. RECOMMENDED PARKING STALL DIMENSIONS FOR VARIOUS TYPES OF STALLS FOR 90° PARKING
(Highway Research Board, 1971)

Type of car	Stall width (ft)	Stall depth (ft)	Aisle width (ft)	Wall to wall distance (ft)	Paved area per vehicle (ft^2)
Small compact[a]	7.5	15.0	20.0	50.0	187.5
Larger American compact[b]	8.0	16.0	22.0	54.0	216.0
Small intermediate[c]	8.0	16.5	22.0	55.0	242.0
Full size	9.5	18.5	25.0	62.0	294.0

[a] Volkswagen and Maverick, for example

[b] Valiant, for example

[c] Dart, for example

Table 28. DIMENSIONS OF 1973 VEHICLES PRODUCED BY FORD MOTOR COMPANY

Vehicle	Length (ft)	Width (ft)	Area/car (ft^2)	Weight (lbs)
1) Full-size Ford	18.3	6.6	121	4,350
2) Full-size Mercury	18.7	6.6	124	4,500
3) Full-size Lincoln	19.2	6.6	127	5,200
4) Intermediate size Torino	17.3	6.6	114	3,800
5) Compact-size Maverick	15.5	5.9	92	2,800
6) Sub-compact Pinto	13.9	5.8	81	2,240

Recent studies have analyzed the feasibility of using porous pavement. Included in this analysis is an estimate of the amount of area which was paved during 1971. The results are shown in Table 29. Note that residential streets are the most important single source for urban areas. If much of this acreage could be paved with porous pavement, a tremendous benefit as far as stormwater quantity and quality control would result. Based on the low rate of utilization of many of these facilities at present, it appears that such modifications might be feasible.

According to the HRB report on parking, the following design criteria are recommended:

1) lots parking five or more vehicles shall be paved with a dustproof all-weather surface;

2) the lot shall be graded so that it will drain as required by the city engineers; and

3) for adequate drainage, a minimum grade of one percent for asphalt or concrete is desirable.

It appears that a reassessment is needed of the tendency to pave the undeveloped portion of commercial and industrial developments. This trend is the result not only of the ever-increasing need for parking but also the desire for reduced on-site maintenance costs. Thus, many service facilities are designed with large amounts of excess capacity for parking. The increased convenience for motorists and owners of such establishments is further aggravating off-site disposal problems. If the appropriate groups could be made aware of these problems, then it would be possible to reduce stormwater management problems and increase the amount of pervious open space in urban areas.

A major transition in attitudes is needed to encourage a re-evaluation of current parking practices. We have become accustomed to nuisance-free parking as one of the positive indicators of quality of life. Unfortunately, such conveniences have caused a significant increase in the intensity of stormwater problems as cities have been paved over in response to this desire for better parking.

Streets

Streets are a major source of sewered runoff from urban areas. Reasons for not permitting drainage water to accumulate on streets are

Table 29. PAVING IN U. S.
(Franklin Institute, 1972)

Type of pavement	Surface area paved during 1971 (10^6 sq. yds.)	
A. Parking lots		46.0
Shopping centers	25.0	
Industry	11.6	
Office buildings	9.4	
B. Residential streets		145.5
Low design (cities ≥ 50,000)	87.5	
Low design (cities < 50,000)	45.9	
High design	12.1	
C. Business streets		57.3
Suburban	52.5	
City	4.8	
D. County road		397.5
Low-volume	193.6	
High-volume	203.9	
E. Highway		246.0
Two-lane	142.0	
Four-lane	104.0	
F. Miscellaneous		75.2
	TOTAL	967.5

numerous, *e.g.*, avoid splashing pedestrians, interference with traffic flow, and so on.

The previous section mentioned that provision of adequate off-street parking is now considered accepted practice in urban areas. Since this is the case, one would expect a re-examination of street design principles in light of these changes. It would seem reasonable to presume that smaller streets might be used if adequate off-street parking is available. Surprisingly, studies of residential areas which relate the two functions are apparently unavailable. A recent report by the National Association of Home Builders Research Foundation, Inc. (1973) addressed this question. They were unable to find any work which related residential street design with the availability of off-street parking. Their conclusions are listed below:

The conclusions listed below are based on results of the National Survey on Street Development Standards. It is intended that these conclusions be used as a basis for discussion and development of recommendations for street development performance standards.

> The Current Situation
>
> 1. There is extensive lack of uniformity of street design standards of cities and counties in the United States.
> 2. Street design standards reflect practices of 20 years or more in the past and have not kept pace with current knowledge.
> 3. Many jurisdictions have not adopted the recommendations of the National Committee for Traffic Safety or the Recommended Practice of the Institute of Traffic Engineers. If they have used these standards as a guide for design values such as street widths, the values often have been increased on a presumably arbitrary basis.
> 4. Current street design standards are often incompatible with the types of streets needed for new patterns of residential development such as planned unit development, cluster

housing and multi-family housing with internal circulation. To a certain extent, this is true even of the two previously mentioned national standards.

5. A number of organizations concerned with street development standards have a highway or major street orientation.

6. The state of the art with respect to designing rigid concrete and flexible asphalt pavement for given traffic load and subgrade conditions is satisfactory to obtain value engineered street pavement sections; for example, the thickness design methods of The Asphalt Institute and Portland Cement Association.

7. There is not widespread acceptance of these design methods by cities and counties. Design procedures, or more likely, rigid specifications of State Highway Departments, are adopted by cities and counties in lieu of establishing their own requirements. These specifications are for heavily traveled thoroughfares and major highways.

8. Regional variations in design and, to a lesser extent, construction standards are not a deterrent to establishing national "model" performance standards.

What Needs to be Done

9. The traditional hierarchy of street types, arterial, feeder, and local streets, needs to be revised and expanded and brought more into line with the performance concept of Average Daily Traffic (ADT). The concept of branching streets should be recognized.

10. Street construction standards should be based on performance principles using the two basic factors of design, ADT or similar index and

standard tests and classifications of subgrade materials, either California Bearing Ratio (CBR), Resistance (R) Value or Bearing Value Determination, in combination with recognized design methods for concrete and asphalt pavements. Pavement design lives should be realistic. Traffic loadings for residential areas should take into account lesser weights of trucks and volume of truck traffic and elimination of future growth factors where the streets do not carry through traffic.

11. There is a need for wider acceptance of proven methods of upgrading street subgrade materials to allow for reduction in pavement surface thickness. There is also a need for further development and experience on new types of economical subgrade treatment.

12. For open-space communities, consideration should be given to distribution of a portion of surface water drainage over ground rather than through streets. This method would allow the elimination of some curb and gutter.

13. There is a need for data on maintenance and repair costs of different pavement types.

14. In general, there is need for an educational program to gain better understanding of the need for value engineered residential street development standards and for new approaches to meeting this objective.

Thus, it appears that current standards for residential street design have evolved on an ad hoc basis with little or no regard for their function. While a dearth of information exists regarding residential street design, there is an abundance of information regarding expressways and major arterial streets (see Paquette *et al.*, 1972, for example). Performance criteria are expressed in terms of traffic flows and related parameters. In fact, benefit-cost type studies have been done but, unfortunately, they are oriented to the traveler and not those who are affected detrimentally by the transportation system.

Interest in streets was nurtured by the observation that, from an environmental point of view, current street design criteria are causing many problems. Exploration began of the possibility of modifying some of these practices. For example, when should curb and gutters be utilized? From the environmental point of view, it is desirable to avoid them if possible. Also, how much inconvenience would result if stormwater inlet design was modified? The trend over the past few decades has been to provide larger and larger inlets and to remove grates. These changes have improved *hydraulic efficiency*, but have aggravated water quality problems.

This research supports the view that streets should be designed on a functional basis that examines all of the benefits and costs and relates the design to needs other than parking and traffic flow. The NAHB study (1973) recommends the standards for residential street design as shown in Table 30.

By contrast with the recommendations shown in Table 30 existing residential streets range in width from 26 feet to 40 feet (8 m to 12 m). The most typical width was 36 feet (11 m) which provides for two 10-foot (3 m) traffic lanes and two 8-foot (2.5 m) parking lanes.

The Denver drainage study classifies streets into the four categories shown below (Drainage Criteria Manual, Vol. I, 1968):

> Local - minor traffic carrier within a neighborhood which usually is characterized by two moving lanes and parking along the curb. There is no through traffic moving from one neighborhood to another. Traffic control is by use of stop or yield signs, and in many cases, there is no formal traffic control.
>
> Collector - collect and distribute traffic between arterial and local streets. There may be two to four moving traffic lanes and parking is allowed adjacent to curbs. Traffic on collectors has right-of-way over traffic from adjacent local streets.
>
> Arterial - permits rapid and relatively unimpeded traffic movement throughout a city. Four to six lanes of traffic exist and parking adjacent to curbs may be prohibited. The arterial traffic normally

Table 30. PROPOSED RESIDENTIAL STREET DESIGN STANDARDS SUMMARY
(NAHB, 1973)

	Street description				
	Place	Lane	Subcollector	Collector	Arterial
Service	Very light	Light	Local traffic	Local & thru	Thru only
Traffic (ADT) (1)	0 - 200	201 - 500	500 - 1,000	1,001 - 3,000	3,000 +
Pavement width					
No parking	18'	18'	26' (2)	28'	(3)
Parking 1 side	18'	18'	28'	36'	(3)
Parking 2 sides	26'	26'	36'	36'	(3)
R of W width	24' to 30'	24' to 30'	44' to 60'	44' to 60'	(3)
Street slope (4)	0.5% to 15%	0.5% to 10%	0.5% to 10%	0.5% to 8%	(3)
Sight distance	75'	150'	200'	250'	(3)
Maximum speed	20 mph	25 mph	30 mph	35 mph	(3)

(1) Average Daily Traffic (ADT) estimates are as follows: single-family detached houses - 7.0; group or townhouses - 6.0; garden apartments 1 to 4 story - 5.0; elevator apartments over 4 stories - 4.0.

(2) Two nine-foot moving lanes plus one eight-foot emergency stopping lane.

(3) Arterial streets shall be designed for specific traffic and roadway conditions as well as other related factors.

(4) Adequate cross slope of at least two per cent is required to prevent ponding.

has the right-of-way over collector streets which it crosses with intersections at approximately 1/4 mile intervals. Construction of an arterial will often include a median strip with traffic channelization and signals at numerous intersections.

Freeway - permits rapid and unimpeded movement of traffic through and around a city. Access to the freeway is completely controlled by interchanges at major arterial streets. There may be up to eight lanes of traffic and parking is not permitted on the freeway right-of-way.

The Denver study defines two storm events for which the facilities should be designed: the initial storm (storm of 2 to 10-year recurrence intervals) and the major storm (storm of 50 to 100-year recurrence intervals). They present detailed engineering design data for analyzing how the drainage system will perform in both of these cases. Traditional drainage system design did not take this integrated approach and analyze the role of the street itself as a stormwater conveyance device. The Denver studies recognize that certain types of streets might be used as a temporary storage areas for these unusual events. Their criteria for minor and major storms are shown in Tables 31, 32, and 33.

For the minor storm, temporary inundation is permitted for the less utilized streets. For the major storm, a deeper inundation is allowed.

The allowable depths were determined based on analysis of the benefits and the costs. The earlier section which discussed parking indicated that many local streets are not utilized efficiently so that sections of the pavement could be made available for temporary storage.

The Denver work is very helpful since the purposes of stormwater quantity and quality control are complementary in this case. The temporary ponding of water would reduce velocities and thereby discourage the transport of some types of pollutants. It would also reduce the peak rate of discharge.

The other major area of concern in this section is street inlets. Two types of inlets are used: curb inlets and gutter inlets. Sometimes a combination inlet is also used. The design of such inlets is discussed in detail in the *Drainage Criteria Manual* (1968). In discussing the pros and cons of inlet design, they state that "The

Table 31. ALLOWABLE USE OF STREETS FOR INITIAL STORM RUNOFF IN TERMS OF PAVEMENT ENCROACHMENT (Drainage Criteria Manual, 1968)

Street classification	Maximum encroachment
Local	No curb over-topping.[a] Flow may spread to crown of street.
Collector	No curb over-topping.[a] Flow spread must leave at least one lane free of water.
Arterial	No curb over-topping.[a] Flow spread must leave at least one lane free of water in each direction.
Freeway	No encroachment is allowed on any traffic lane.

[a] Where no curbing exists, encroachment shall not extend over property lines.

Table 32. MAJOR STORM RUNOFF ALLOWABLE STREET INUNDATION

Street classification	Allowable depth and inundated areas
Local and collector	Residential dwellings, public, commercial, and industrial buildings, shall not be inundated at the ground line, unless buildings are flood proofed. The depth of water over the gutter flowline shall not exceed 18 inches.
Arterial and freeway	Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line, unless buildings are flood proofed. Depth of water at the street crown shall not exceed 6 inches to allow operation of emergency vehicles. The depth of water over the gutter flowline shall not exceed 18 inches.

Table 33. ALLOWABLE CROSS STREET FLOW

Street classification	Initial design runoff	Major design runoff
Local	6-inch depth at crown or in cross pan	18 inches of depth above gutter flowline
Collector	Where cross pans allowed, depth of flow shall not exceed 6 inches	18 inches of depth above gutter flowline
Arterial	None	6 inches or less over crown
Freeway	None	6 inches or less over crown

curb inlet does not clog readily which is its major advantage. The large dimensions of the clear opening compared to that of a grated inlet allows trash to pass into the storm sewer system rather than be trapped at the inlet."

It is at this point that conflicts arise between the desire for greater hydraulic efficiency and the water quality management goal of keeping the "trash" out of the storm sewer. Table 34 shows the proportion of theoretical capacity which can be used as a function of the type of inlet. As can be seen, the grated inlets are significantly less efficient from a hydraulic point of view. They are more effective from a water quality viewpoint. By adopting the design using larger curb inlets, road engineers have reduced their costs substantially. Unfortunately, this has further aggravated water quality problems. For example, the current gutter design in Gainesville, Florida uses an open curb gutter with a length of several feet. Such inlets have a very high hydraulic capacity so that the water drains rapidly. The design of inlets is an example of where people could, by proper housekeeping, use more restrictive inlets if they are cleaned properly. The consequences of clogged inlets are minor for most residential streets and they accrue to people who are responsible for the clogged inlets in the first place.

This study of streets has brought out the fact that it may be possible to modify the design of, or eliminate curb and gutters in lower density residential areas with well-drained soils. A significant component of land use in urban areas is for streets and associated right-of-way. For example, the typical street in Chicago is 34 feet (10.4 m) wide with a right-of-way of 66 feet (20 m). Residential streets in Gainesville, Florida are 28 feet (8.5 m) wide with a 50 foot (15 m) right-of-way. In both of the above examples, curb and gutter is used. As will be shown later in the economics section, the use of curb and gutter with associated storm drains about doubles road construction costs.

In many residential areas of Gainesville, curb and gutter is not used. Instead, the street drains onto adjacent vegetated swales which eventually drain to the receiving water.

From a water quality point of view, it is highly desirable to avoid curb and gutter since the natural drainage pattern will encourage infiltration and the vegetation serves as a water quality control device.

Table 34. REDUCTION FACTORS TO APPLY TO INLETS (Drainage Criteria Manual, 1968)

Condition	Inlet type	Percentage of theoretical capacity allowed
(1)	(2)	(3)
Sump	Curb opening	80%
Sump	Grated	50%
Sump	Combination	65%
Continuous grade	Curb opening	80%
Continuous grade	Deflector	75%
Continuous grade	Longitudinal bar grated	60%
Continuous grade	Transverse bar grate or longitudinal bar grate incorporating transverse bar	50%
Continuous grade	Combination	110% of that listed for type of grate utilized

The *Drainage Criteria Manual* (1968) gives permissible velocities for various types of vegetation, soil types, and slopes. This report recommends design velocities of ≥ 2 ft/sec (0.6 m/sec) for initial runoff to minimize sediment depositional problems. Recommended design depths should be less than 1.5 feet (0.46 m) and preferably less than 1.0 feet (0.3 m). This report takes exception to the minimum velocity constraint for such roadside channels. It is the opinion of the writers that it is preferable to design the channel such that sediment transport does not occur during the minor storm.

In summary, it appears that one could design, in many parts of urban areas, aesthetically appealing and yet functionally sound drainage systems which avoid the use of storm sewers.

Rooftops

Basically, there are two types of rooftop design in common use, *i.e.*, flat roof and gabled roof. In residential areas, buildings up to three stories in height tend to have gabled roofs. Flat roofs on the other hand, are almost entirely used for residential buildings more than three stories in height and for non-residential construction such as warehouses, shopping centers, etc. Flat roofs are less expensive to build than the gabled roofs.

Roofs are designed to carry a specified live load in addition to their own weight. Many building codes, especially in the northern states, also require roofs to be designed for a specified snow load.

From the standpoint of utilizing the roof as a potential storage area during runoff, it is impossible to utilize gabled roofs. Flat roofs are therefore the only type that can be considered for such a use. However, it must be recognized that the life of a flat roof used for storing water might be less than that of gabled roof or even flat roof without storage.

The amount of water that can be stored on an existing flat roof will depend on the design load. New rooftops can be designed to store larger amounts of water.

E. ECONOMIC ANALYSIS OF CONTROL OPTIONS

This section presents cost data associated with various control alternatives. The analysis is partitioned into three sections:

storage options and costs, treatment options and costs, and miscellaneous options and costs.

1. *Storage Options and Costs*

Basically, there are five storage options:

1) surface storage such as reservoirs, ponds, tanks, etc.;

2) in-system storage such as in trunk lines and interceptors;

3) underwater storage;

4) deep tunnels; and

5) underground modules.

The main objective behind all these options is to provide temporary storage of combined overflow during peak storm runoff periods and to release this flow to the treatment facility during subsequent periods of lower flow. Costs for various kinds of storage facilities based on Sandusky, Ohio data are given in Table 35. A comparison of storage costs for several cities is shown in Table 36. Note the wide variation in costs which indicates the significance of local conditions.

2. *Treatment Options and Costs*

A variety of treatment options and costs are available in the original SWMM. As a consequence of this research, several other treatment options have been incorporated in the SWMM Model as in Section IV-D. The cost functions of other treatment options are described herein. In addition, several cost functions in the original SWMM have been revised in the light of data recently made available.

Cost data that have been made available recently pertain to the following units:

1) high-rate filtration;

2) microstrainers; and

3) dissolved air flotation.

Table 35. COST OF STORAGE IN SANDUSKY, OHIO (Rohrer and Associates, Inc., 1971)

Storage method	Storage capacity in gallons	Construction cost[a] in dollars
Reinforced concrete tank	200,000	84,930
	1,000,000	358,670
Lagoons	200,000	95,820
	1,000,000	351,740
Underwater tank	200,000	22,000
	1,000,000	102,000

[a] Excluding engineering, land and acquisition costs.

Table 36. COMPARISON OF STORAGE COSTS FOR VARIOUS CITIES
(Rohrer and Associates, Inc., 1971)

	Area served in acres	Cost $\$x10^6$	Cost/acre \$/acre	Cost/capita \$/capita
Sandusky, Ohio Underwater storage (1 year storm)	580	3.57	6,160	---
Sandusky, Ohio Underwater, underground and surface (1 year storm)	2,205	7.75	3,500	212
Chicago, Illinois Deep tunnel (100 year storms)	192,000	1,270	6,615	423
Saginaw, Michigan	-------	--------	415 - 5,140	62 - 670
Chippewa Falls, Wisconsin Lagoon	90	.61	6,780	---
Other cities	-------	--------	20 - 6,500	6 - 1,300

The following new treatment options have been added to the Model:

1) swirl regulator/concentrator;

2) biological treatment; and

3) high-rate disinfection.

Other treatment options utilize the cost functions provided in the original SWMM (EPA, 1971a).

Revised Cost Functions

High-Rate Filtration -- The original Model computes the capital cost, in dollars, of this unit using the following equation:

$$\text{Capital Cost} = 54{,}000\ Q^{0.67}\left(\frac{\text{ENR}}{1058}\right)F \qquad (15)$$

where Q = capacity, mgd

ENR = Engineering News Record (ENR) index for prescribed year

F = ENR site factor for the geographic location of treatment facilities

Recent pilot plant studies conducted by Hydrotechnic Corporation (1972) on high-rate filtration outlined cost data which are shown on Figure 32. On the same figure, the cost that would be computed by the Equation (15) is also plotted for ENR of 1470 and a site factor of 1.0. Hydrotechnic Corporation developed cost data for two filtration rates, *i.e.*, 16 gal/min/ft^2 (65 cm/min) and 24 gal/min/ft^2 (98 cm/min).

It can be seen that Equation (15) would predict a lower cost than estimated by Hydrotechnic Corporation (1972). Realizing that various uncertainties exist, it appears appropriate to use the following equation for cost purposes:

$$\text{Capital Cost} = 85{,}000\ Q^{0.67}\left(\frac{\text{ENR}}{1058}\right)F \qquad (16)$$

The curve derived from this equation for ENR of 1470 and a site factor of 1.0 is also shown in Figure 32. It is evident that this curve gives a better estimate than the original one. It has therefore been incorporated in the SWMM.

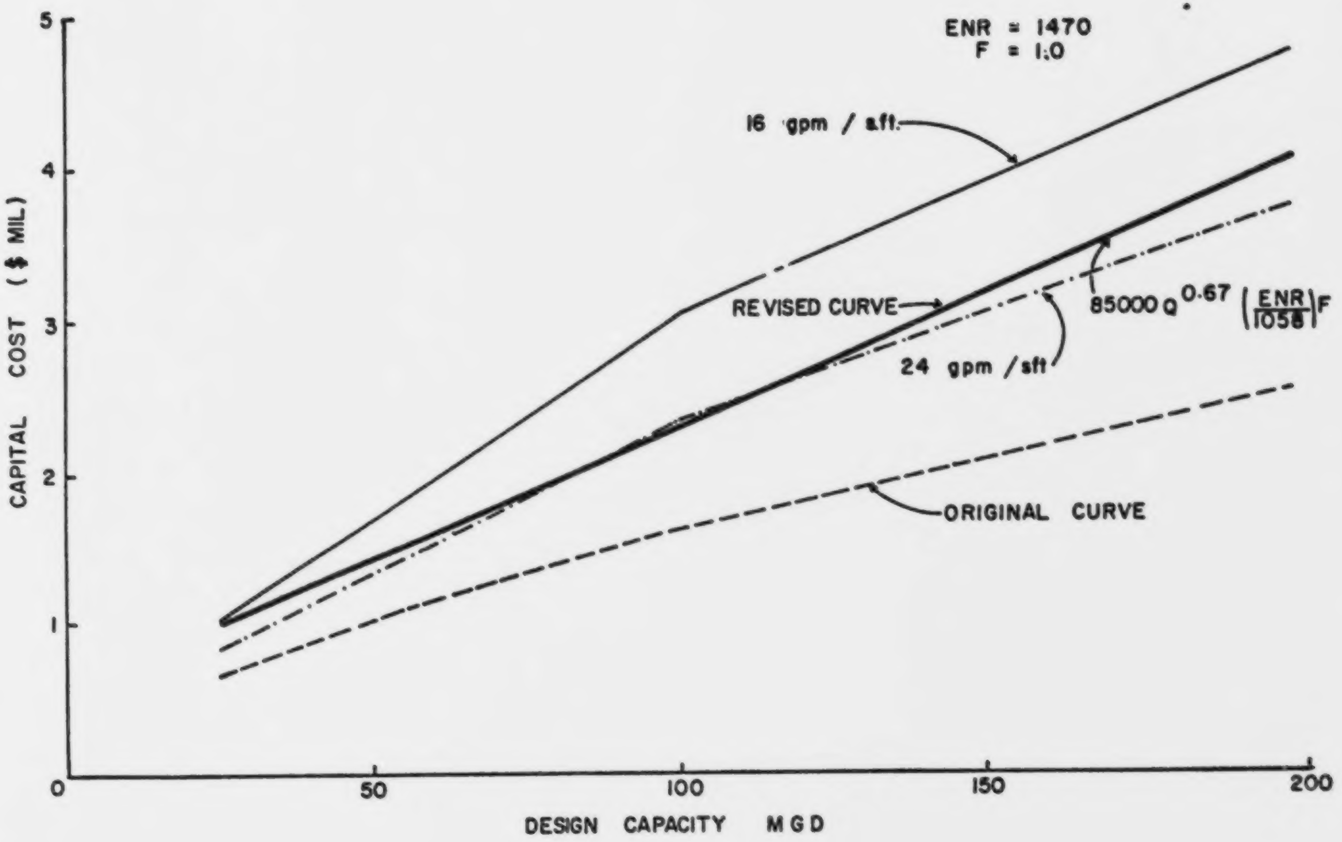


FIGURE 32.
Capital Cost for High-Rate Filtration.

Microstrainers — The original Model computes the capital costs of the microstrainer facility based on one of the following equations:

$$\begin{aligned} \text{Capital Cost} &= 30{,}000\ Q\left(\frac{ENR}{1314}\right)F \quad \text{for } Q \leq 25 \text{ mgd} \\ &= 20{,}000\ Q\left(\frac{ENR}{1314}\right)F \quad \text{for } Q \geq 25 \text{ mgd} \end{aligned} \qquad (17)$$

Cochrane Division of Crane Company has conducted studies on microstrainer installations (Glover, 1972). According to Glover's estimate, a 30 mgd (1.3 m^3/sec) microstrainer installation in 1969 would cost \$217,000. This gives a cost of \$7,200/mgd. A comparison of this cost with the equations in the original Model indicates a considerably higher cost predicted by the Model. It is proposed to use the following relationship which corresponds to Glover's work:

$$\text{Capital Cost} = 10{,}000\ Q\left(\frac{ENR}{1383}\right)F \qquad (18)$$

Dissolved Air Flotation -- Capital costs for this unit are computed in the original Model by one of the following equations:

$$1.35\ Q \exp\left(\frac{2.3026}{0.2075 + 0.0114 \log Q}\right)\frac{ENR}{1098}F \quad \text{for } Q \leq 15$$

$$1.35\ Q \exp\ [2.3026(4.6032 - 0.0559 \log Q)]\ \frac{ENR}{1098}F \quad \text{for } 15 \leq Q \leq 100 \qquad (19)$$

$$31{,}000(1.35)Q\left(\frac{ENR}{1098}\right)F \quad \text{for } Q \geq 100$$

Full-scale units are currently in operation in Milwaukee with a design flow of 5 mgd and in Racine, Wisconsin with design flows of 14.13 and 44.4 mgd (0.629 and 1.95 m^3/sec) each. The construction costs of these facilities have been reported to be about \$25,000/mgd in 1971 (Rex Chainbelt, 1972).

An examination of the model equations outlined above indicates that capital costs estimated are substantially higher than reported for Milwaukee and Racine installations. The following equation is therefore proposed for all capacities:

$$\text{Capital Cost} = 31{,}000\ Q\left(\frac{ENR}{1400}\right)F \qquad (20)$$

New Treatment Options

Swirl Concentrator -- A full-scale, 36 foot (11 m) diameter installation is currently under construction at Lancaster, Pennsylvania. Cost of this installation has been estimated at $100,000 and is expected to handle about 165 cfs (4.7 m^3/sec) of flow. Based on the above data, it is proposed to estimate the cost using:

$$\text{Capital Cost} = \frac{100{,}000}{36} D \left(\frac{ENR}{1800}\right) F \qquad (21)$$

where D = diameter of the swirl concentrator, ft.

Following construction of this and other installations, a revised equation will be utilized.

Biological Treatment -- Cost of this unit is based on the full-scale contact stabilization plant now in operation in Kenosha, Wisconsin, with a capacity of 20 mgd (0.88 m^3/sec). Cost of this facility has been estimated at $55,000/mgd (Agnew, 1973). Thus, the following equation is proposed for the purpose of modeling the cost of this unit:

$$\text{Capital Cost} = 55{,}000\ Q \left(\frac{ENR}{1400}\right) F \qquad (22)$$

High-Rate Disinfection -- Cost modeling of this unit is based on studies conducted by Cochrane Division of Crane Company (Glover, 1972). Cost of a 30 mgd (1.3 m^3/sec) facility is estimated at $59,000. Thus, the cost equation proposed for modeling this unit is as follows:

$$\text{Capital Cost} = 2{,}000\ Q \left(\frac{ENR}{1314}\right) F \qquad (23)$$

Other Treatment Options -- Cost data for other options already in the Model remain the same. Many other treatment options are currently under investigation (Field and Struzeski, 1972). Enough information is not available at this time to justify their inclusion in the Model.

3. *Miscellaneous Options and Costs*

Porous Pavement

The primary objective of porous pavement is to reduce runoff from paved areas, by allowing rain to percolate through it, thereby reducing the magnitude of outfall discharges. In its simplest form, the porous pavement consists of a base course of crushed gravel with a surface course of porous asphalt concrete. In a recent publication by The Franklin Institute Research Laboratories (1972), data are presented on the depth of the surface and base course for various kinds of uses. In considering the use of porous pavements, consideration of features such as geographical area, temperature, subsurface and soil conditions, and possibility of groundwater contamination are important. A cost comparison between conventional and porous pavement for various uses (The Franklin Institute, 1972) is given in Table 37.

This tabulation shows that the costs for porous pavement compare favorably with conventional pavements. It is pointed out that, at present, there is only one full-scale installation near Houston, Texas that recently started evaluating porous pavements for flow attenuation and therefore its feasibility has yet to be demonstrated.

Erosion Control Options

Erosion from land which has been partially or completely denuded is most often controlled with the use of various on-site methods. These methods generally fall into two classes; namely, structures and stabilizing treatments. In Table 38, the costs for various erosion control alternatives are listed (Brandt *et al.*, 1972).

There are generally two types of chemicals that have been developed for control of erosion. These two types are listed in Table 38 as Chemical A and Chemical B. Chemical A has a manufacturer's price of about $150/acre ($370/ha) and usually is applied with 2,000 gallons of water/acre (18,700 l/ha). Chemical B has a price of about $250/acre ($618/ha) and is applied with 1,000 or less gallons/acre (9,350 l/ha) (Brandt *et al.*, 1972).

Table 38 gives the costs involved in procurement and application of the control and does not include design or maintenance costs. Maintenance costs can vary considerably and consequently, it is difficult to give a general cost estimate. Where no real estimates are available, a figure of $125/acre ($309/ha) for maintenance during the developmental process has been used by some urban developers. It has been estimated that cleaning sediment basins costs $2.50/ton of sediment removed (Brandt *et al.*, 1972).

4. *Options for Removal of Surface Pollutants*

Cost estimates for alternatives which involve the removal of surface pollutants from the drainage system such as street cleaning, catchbasin cleaning, and sewer flushing can usually be obtained from the local public works department for the area of concern. Where no such estimates are available, the costs listed in Table 39 can be used.

Table 37. COMPARISON OF THE COST OF CONVENTIONAL PAVEMENT AND POROUS PAVEMENT
(Franklin Institute, 1972)

Type of use	Porous pavement		Conventional pavement	
	Cost range $/ yd^2	Average cost $/ yd^2	Cost range $/ yd^2	Average cost $/ yd^2
Parking lot	6.32 - 6.40	6.36	3.65 - 5.81	4.73
Residential street				
Low design	6.33 - 6.37	6.35	4.80 - 9.60	7.20
High design	6.19 - 6.56	6.38	8.35 - 12.10	10.23
Business street				
Suburban development	6.65 - 6.97	6.78	6.90 - 9.90	8.40
City	6.99 - 8.32	7.66	14.20 - 22.20	18.20
County road	6.01 - 8.32	6.44	7.24 - 13.23	9.20
Highway - two-lane	6.80 - 7.92	7.36	8.54 - 20.45	14.50

Table 38. EROSION CONTROL COSTS
(Brandt *et al.*, 1972)

A) Structures	
1) Diversion berms	$ 1.25/linear ft
2) Interceptor berms	$ 1.25/linear ft
3) Sodded ditches	$ 4.50/linear ft
4) Grade stabilization structure	$ 2.80/linear ft
5) Level spreader	$ 2.80/linear ft
6) Small sediment basin	$ 500.00/each
B) Stabilizing treatments	
1) Seed, fertilizer, mulch	$1,500.00/acre
2) Seed, fertilizer, straw tack	$ 493.00/acre
3) Seed	$ 209.00/acre
4) Chemical A	$ 300.00/acre
5) Chemical B	$ 387.00/acre
6) Seed, fertilizer, chemical A	$ 480.00/acre
7) Seed, fertilizer, chemical B	$ 567.00/acre

Table 39. UNIT COSTS OF CATCHBASIN CLEANING, SEWER CLEANING AND MECHANICAL STREET SWEEPING (APWA, 1969)

Catchbasin cleaning	
Hand	\$ 3.29/catchbasin
Eductor	\$ 3.47/catchbasin
Orange peel	\$ 4.38/catchbasin
Sewer cleaning	
Flushing	\$ 8.12/1,000 linear ft
Scraping	\$187.00/1,000 linear ft
Mechanical	
Street sweeping	\$100.00/shift

These costs were estimated from 1967 data collected in Chicago, Illinois (APWA 1969).

F. SPECIFICATION OF PERFORMANCE CRITERIA

This report provides a method for examining ways to control the water quality associated with urban stormwater. However, the water quality problem is intimately related to the corresponding problem of controlling water quantity. Most of the control alternatives have a direct effect on quality as well as quantity. For example, stormwater retention basins are growing in popularity as devices for controlling runoff rates (Poertner, 1973). Such basins may also provide sediment control. Unfortunately, the retention basin design may not have considered water quality control and the result can be eyesore for the neighbors. Recognizing the interrelationship, the problem has been addressed from the point of view of designing control facilities for stormwater quantity and quality management.

It appears that the "design event" for control of quantity and quality would often differ. Typically, a higher recurrence interval would be used for water quality control. There are several justifications for this assumption:

1) A prime concern in water quality control is the "first flush" associated with the transport of pollutants during early parts of the storm.

2) As the frequency of the storm decreases, the effect of pollutants being washed into the sewer system diminishes in relative importance and natural processes become dominant.

3) The "pollutants" associated with these natural processes are comprised primarily of sediment and vegetation.

4) For more significant storms, the flow in the receiving water is increased substantially.

Studies in Denver provide a thorough examination of urban stormwater problems. They specify design criteria for two types of storms: minor storms and major storms. The former storms are designed around drainage criteria whereas the latter storm is designed for flood control. Design criteria for the minor storms are shown in Table 40. Design criteria for the major events are shown in Table 41.

Table 40. LEVELS OF PROTECTION FOR MINOR STORM EVENTS
(Drainage Criteria Manual, Vol. I, 1968)

Land Use	Recurrence interval (years)
Residential	2
High-value general commercial area	5
Public building area	5
Airports	2-5
Major airport terminals	5-10
High-value downtown businesses	5-10

Table 41. LEVELS OF PROTECTION FOR MAJOR STORM EVENTS
(Drainage Criteria Manual, Vol. I, 1968)

Land use	Recurrence interval (years)
Urban	100
Industrial	Varies
Agriculture	
High-value	10-25
Low-value	2-10

The purpose of discussing both the minor storm and major storm criteria together is to emphasize to the reader that these are separate drainage systems with different purposes. The purpose of the design for the minor system is ". . . to prevent inconvenience and frequently recurring damage caused by frequent storm runoff" (*Drainage Criteria Manual*, 1968). The purpose of the major system is to control flooding with its associated significant and sometimes catastrophic consequences.

Urban drainage systems have been designed based on the minor storm with little or no consideration as to what will happen during the major storm. It may be that designers felt that the design for the minor storm would also provide improved flood control during the major storm. Unfortunately, just the opposite is often the case. The work in Denver represents an excellent start towards inducing designers to analyze the performance of the system for both the *minor event and the major event*. For example, streets play a very important role as conveyance channels for stormwater. In the Denver report, criteria are presented for allowable flow along various types of streets for the minor and major storms (see Tables 31 to 33 in this section).

With regard to stormwater quality management, the design criteria we recommend are similar to the criteria for the minor storm. As shown earlier, urbanization causes higher peak flows due to the increased impervious areas which discharge directly to storm sewers, roadside ditches, etc., and thence into the receiving water. In the following sections, a discussion is presented regarding the selected performance criteria.

These proposed criteria attempt to physically internalize the externalities caused by stormwater runoff using a quasi-closed system approach. It will be argued that owners of land within the urban area are responsible for "safe" disposition of stormwater. This may be achieved on-site by discharging stormwater onto pervious areas and/or retention basins within each parcel of property; or by forming a coalition with other owners to accomplish this same purpose off-site by acquiring a common retention basin, flood plain, etc., for this purpose. Off-site control could be on publicly owned land which could serve also as a park, recreational area, etc. This approach has the potential advantage of providing an explicit procedure for justifying open spaces within urban areas by recognizing the vital function it serves in water resources management. By physically internalizing the externality, one has partially resolved the equity question since the urban property owners are comparing control costs *vs* on-site damage costs. Thus, they are recipients of the damages if the system is not designed and operated properly, not someone downstream. A similar argument might be made for other types of developments.

Natural systems cause less off-site effects than developed lands because of a longer detention time for stormwater which permits additional infiltration of water. Also, the vegetation and soil serve as water quality control devices. The approach taken in this section assumes that the owner of land is responsible for safe disposition of stormwater from an environmental point of view. The design event is assumed to be the same as that shown in Table 40. It will be argued that the stormwater from this event and more frequent events will be handled such that there is:

1) no increase in the peak flow observed prior to development; and

2) adequate treatment of stormwater prior to release to the receiving environment.

Adequate treatment may be achieved by utilizing an appropriate control device such as percolating the stormwater through the soil, or providing a retention basin and control device with a sufficient residence time. The responsible individual or group may provide these facilities on his own property and/or form a coalition with others to accomplish the same purpose at a centralized facility.

These same principles can be applied to related off-site control alternatives. Two extensions of the on-site analysis are needed, *i.e.*, the transport cost for directing the water to the control facility; and the cost of the control facility itself. The control facility may be a pervious area such as a flood plain or a structural facility such as a high-rate treatment device. The same performance criterion can be applied as was done in the case for individual on-site treatment. An advantage of this approach is that responsibility for the stormwater can be assigned which is essential to an equitable apportionment of control costs.

Similar performance criteria are being used by some cities for stormwater quantity management. For example, the Metropolitan Sanitary District of Chicago requires control of stormwater according to the following criteria (personal communication, 1973):

1) A combination of storage and controlled release of stormwater runoff is required for all commercial developments exceeding 5 acres in area, for all residential developments exceeding 10 acres in area and for all residential developments between 5 and 10 acres which have an imperviousness of 60 percent or greater. How-

ever, a residential development of 10 acres or less must have an adequate outlet as signified on the District permit by the Municipal Engineer. If the outlet is not adequate, then detention, as determined by the Municipal Engineer, will be required to store that portion of the runoff exceeding the outlet capacity.

2) The release rate of stormwater from all developments requiring detention shall not exceed the stormwater runoff rate from the area in its natural undeveloped state.

3) Drainage systems shall have adequate capacity to bypass through the development the flow from all upstream areas for a storm of design frequency assuming that the land is in a fully developed state under present zoning or zoning proposed under a Comprehensive Plan. The bypass flow rate shall be computed utilizing a runoff coefficient of not less than 0.35. An allowance will be made for upstream detention when such upstream detention and release rate has previously been approved by the District and that evidence of its construction can be shown.

4) The live detention storage to be provided will be calculated on the basis of the 100-year frequency rainfall as published by the U. S. Weather Bureau for this area. The detention volume required will be that necessary to handle the runoff of a 100-year rainfall, for any and all durations, from the fully developed drainage area tributary to the reservoir, less that volume discharged during the same duration at the approved release rate.

G. SYSTEM OPTIMIZATION AND EXAMINATION OF EQUITY QUESTION

The objective of this analysis is to devise a decision-making model for the specific problem of urban stormwater quality management. A fundamental question is to determine the optimal level of pollution control. There are basically two approaches. One is to assess the damages caused by the pollutant, convert them into an equivalent monetary value, and thereby determine an aggregate receptor damage function. Knowing this function and the cost of controlling the pollution, one can determine the optimal degree of control. The primary problem with this approach is that it is extremely difficult, if not impossible, to quantify the receptor damage function. The other approach is based on specifying performance standards which are based on a review by experts of the technical, economic, financial, social and political aspects of the problem. Given this set of standards, the pollutors are assumed to select the least costly way of satisfying that standard. This is the approach we shall use. The selected performance standards were discussed earlier.

Thus, the stated objective for this analysis is to determine the least cost solution subject of meeting a prescribed set of performance standards. This least cost solution might comprise a wide variety of on-site controls as well as off-site controls. However, the solution to the optimization problem is not the final answer. In addition, we need to analyze whether the solution is also fair to everyone. This question will be examined using recent developments from cooperative N-person game theory.

There are numerous ways to control pollution using on-site control and/or off-site control. Thus, a general framework is needed for addressing the problem. The selected approach is based on the planning theory of zoning (Herzog, 1969). Using the planning theory of zoning, each source of pollution calculates the cost of handling the problem on-site as a function of the allowable rate of release from his area. The cost function could look like the curve shown in Figure 33.

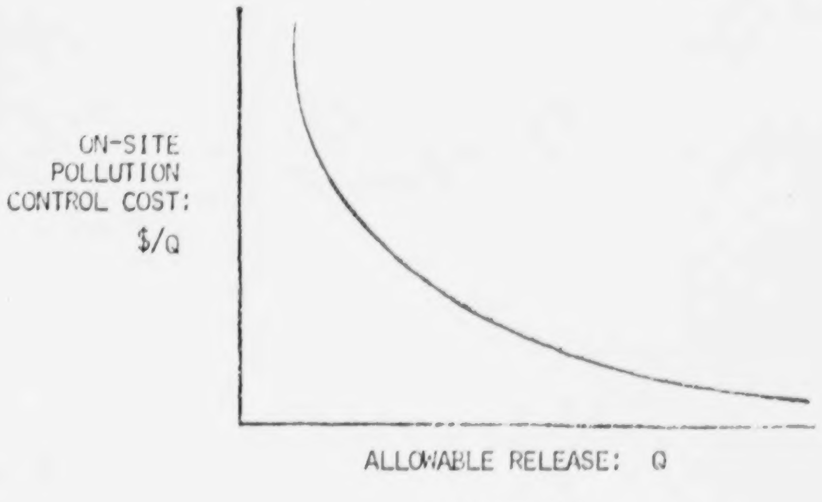


FIGURE 33.
Cost Function for On-site Stormwater Control

Costs decrease as the allowable release increases. In fact, the cost falls to zero as he is allowed to release more and more pollution. Assume that such a curve exists for each area. Note that the abscissa could be air pollutants, water pollutants, noise or any other "nuisance" which is normally covered by zoning regulations.

In this case, the policy question is to determine how much pollution can be released from each of these areas. Using the planning theory of zoning, the answer depends on a determination of "assimilative capacity." Assume that the required level of control can be specified.

The following example is presented in order to illustrate many of the concepts to be discussed. Assume the city under consideration has been partitioned into three study areas. Each study area has two options:

1) on-site control; and/or

2) off-site control at a central storage facility.

The following notation is used:

Let x_{ij} = decision variable: number of units of control j selected for area i

$\bar{x}_{ij}$ = upper bound on x_{ij}

c_{ij} = unit cost for control j in area i

D_i = quantity of commodity originating in area i

$\bar{Q}_i$ = maximum allowable release of stormwater from area i

Z_i = total control cost to area i

π_i = reduction in control cost to area i if $\bar{Q}_i$ is increased by one unit

t_i = unit cost of transporting water from area i to the central control location

c = unit cost of central control

$\bar{W}$ = maximum available control at central facility

The example problem is shown in Figure 34 using a network representation. This problem is deliberately oversimplified to permit us to understand the concepts without getting enmeshed in computational difficulties. The results can be extended easily to more realistic cases where multiple central control facilities exist.

The overall objective function seeks to minimize the total cost of on-site and off-site control. The problem facing each study area is to minimize

$$Z_i = \sum_j c_{ij}\, x_{ij} + (c+t_i)Q_i$$

subject to

$$\begin{aligned} &\sum_j x_{ij} + Q_i = D_i \\ &Q_i \leq \bar{Q}_i \qquad\qquad (24) \\ &x_{ij} \leq \bar{x}_{ij} \quad \text{for all } j, \\ &x_{ij} \geq 0 \quad \text{for all } j, \text{ and} \\ &Q_i \geq 0 \end{aligned}$$

For area 1, his problem is to select x_{11}, x_{12}, and x_{13} so as to

minimize $Z_1 = 10x_{11} + 5x_{12} + 0x_{13} + 4Q_1$

$$\begin{array}{lllllr} \text{subject to} & x_{11} + & x_{12} + & x_{13} + & Q_1 = & 500 \\ & & & & Q_1 \leq & \bar{Q}_1 \\ & x_{11} & & & \leq & 100 \\ & & x_{12} & & \leq & 200 \\ & & & x_{13} & \leq & 200 \\ & & x_{11}, x_{12}, x_{13}, Q_1 \geq 0 & & & \end{array}$$

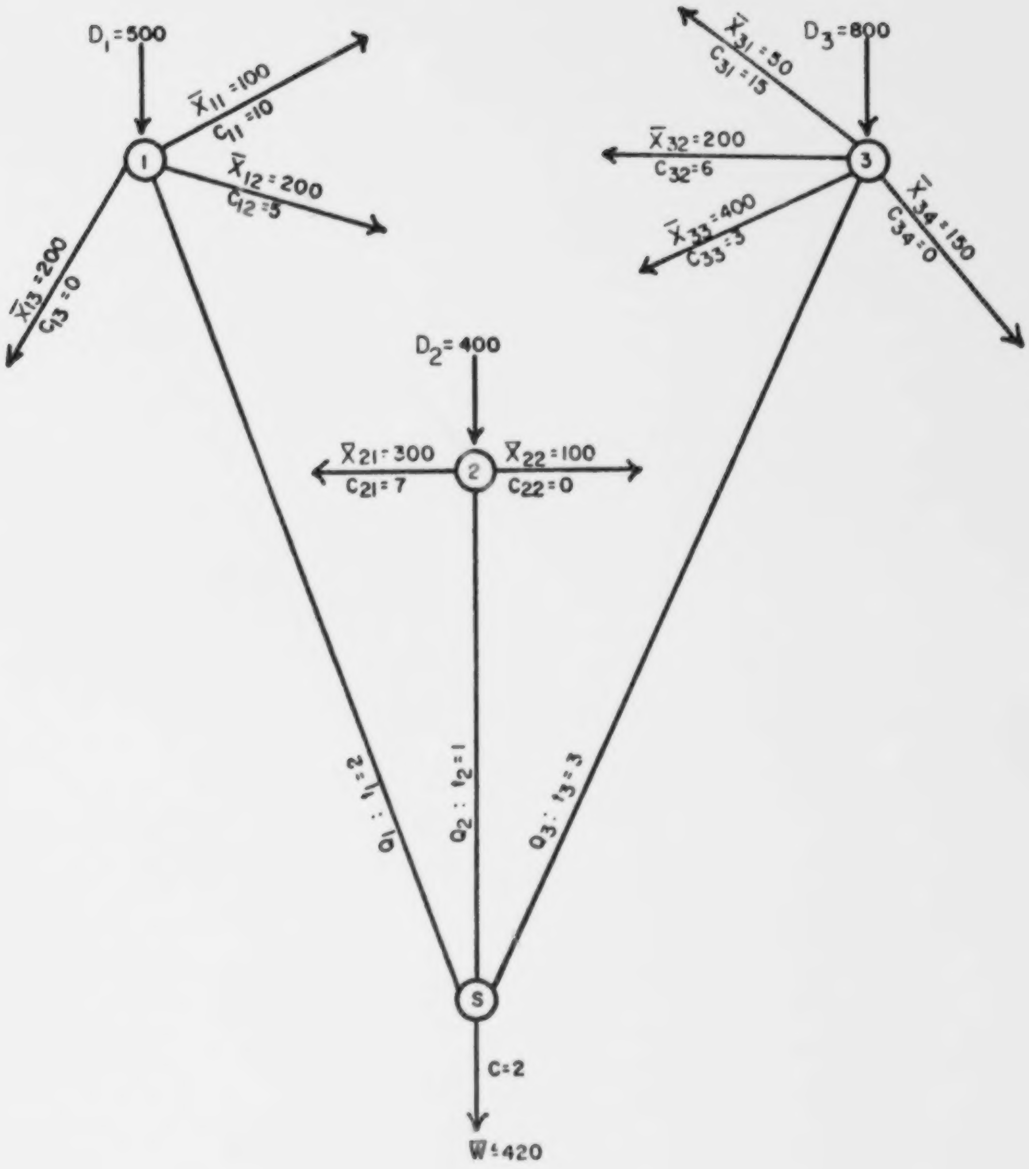

$D_1 = 500$
$\bar{X}_{11} = 100$
$C_{11} = 10$
$\bar{X}_{12} = 200$
$C_{12} = 5$
$\bar{X}_{13} = 200$
$C_{13} = 0$
1
$D_3 = 800$
$\bar{X}_{31} = 50$
$C_{31} = 15$
$\bar{X}_{32} = 200$
$C_{32} = 6$
$\bar{X}_{33} = 400$
$C_{33} = 3$
$\bar{X}_{34} = 150$
$C_{34} = 0$
3
$D_2 = 400$
$\bar{X}_{21} = 300$
$C_{21} = 7$
$\bar{X}_{22} = 100$
$C_{22} = 0$
2
$Q_1 : t_1 = 2$
$Q_2 : t_2 = 1$
$Q_3 : t_3 = 3$
S
$C = 2$
$\bar{W} = 420$

FIGURE 34.
Network Representation of Example Problem.

This is a very simple problem to solve if $\overline{Q}_1$ is known.

As it turns out this is a question of critical importance in some cases. In the context of stormwater quality management, $\overline{Q}_1$ represents a judgment on the part of the administrator as to the "assimilative capacity" or availability of off-site controls to area 1. Traditionally, the natural system has provided these off-site controls free of charge. For example, the off-site area in this example could be a swamp or flood plain.

In order to find $\overline{Q}_1$, we need to know how much each area is willing to pay for off-site control *based on their alternative on-site control costs*. This can be done by solving the above linear program for various values of Q_i assuming on-site control is required. Thus, for the moment, Q_i is analogous to an effluent standard imposed on area i. The problem can be solved by deleting the last term, $(t_i + c)Q_i$, from the objective function and finding the optimal solution for assumed levels of Q_i. Computationally this can be done quite easily using parametric programming as explained below. Initially, set $Q_i = 0$ and solve the linear programming problem. Then, as a post-optimal procedure, one can vary Q_i continuously from 0 up to any prescribed upper bound using as the right hand side $Q_i = 0 + \Theta r$ where Θ equals a parameter which will increase continuously from 0 to 1 and r is a scalar, say 1000, in this case. The solution to this problem tells the total cost to area i for any value of Q_i which is of interest. This is a very attractive feature of linear programming. Another aspect of linear programming which is of interest is duality theory. The solution to the dual problem is obtained when the above problem (primal problem) is solved. Among other things, it tells, for a given Q_i, the reduction in cost to area i if Q_i is increased by one unit. This unit cost is called the "shadow price" with respect to Q_i and will be denoted as π_i.

Let us solve the example problem for area 1 assuming $\overline{Q}_1 = 0$. The answer is simply that on-site control is used. As the constraint on Q_1 is relaxed, area 1 will substitute off-site control for its most expensive on-site control, x_{11} in this case. Thus, it is saving \$6 per unit change in Q_1 in this range. The analysis continues in this manner until all solution possibilities have been identified. The results are shown in Figure 35.

Assume that a similar analysis was done for areas 2 and 3. Then the willingness to pay of each of the three areas would be known. Next assume each area was offered as much storage as they wanted at its cost of $(t_i + c)$ dollars per unit. In this case, the aggregate demand would be $Q_1 = 300$, $Q_2 = 300$, $Q_3 = 250$, or a total demand of 850 units. But suppose only 420 units are available. How do we

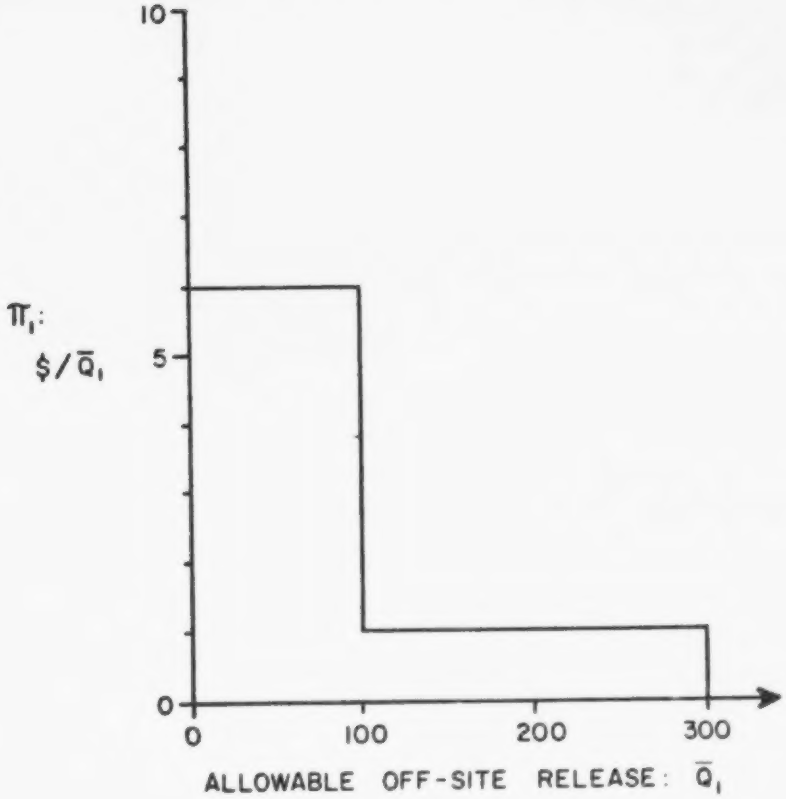


FIGURE 35.
Shadow Price for Area 1 for Assumed Values of $\overline{Q}_1$.

allocate the available storage? If you are the coordinator or planning authority, we assume that you will attempt to maximize the aggregate savings to area 1, 2, and 3 from using the regional facility. Alternatively preference will be given to those areas whose on-site control costs are the highest.

The answer to the above question can be obtained by solving one larger optimization model which is formulated on the following page as Table 42. The constraints for this problem can be divided into two categories:

1) three sets of study area constraints; and

2) a coupling constraint.

The coupling constraint is the only linkage among the three study areas. If the values of Q_1, Q_2, and Q_3 are prespecified, then the larger problem can be completely decomposed into three independent subproblems. There are many real-world situations in which an *a priori* apportionment is used, *i.e.*, allocation to each area based on its size, population, etc. For a pollutant, $\overline{W}$ might be the assimilative capacity of the receiving water which has been apportioned. In the case of a pollutant, the appropriation is equivalent to prescribing effluent standards.

Assume an apportionment is established such that $Q_1 = Q_2 = Q_3 = 420/3$. Given this apportionment, find Z_1, Z_2, and Z_3, the least cost solutions for the three areas. The results are:

$$Z_1 = \$1360 : \pi_1 = \$1 : \qquad Q_1 = 140$$

$$Z_2 = 1540 : \pi_2 = 4 : \qquad Q_2 = 140$$

$$Z_3 = \underline{2560} : \pi_3 = 1 : \qquad Q_3 = \underline{140}$$

$$\sum_{i=1}^{3} Z_i = \$5460 \qquad \sum_{i=1}^{3} Q_i = 420$$

Apportioning the water among the three areas results in a combined cost of $5,460. Next we will examine whether, from a least cost point of view, it would be possible to select Q_1, Q_2, and Q_3 such that costs are reduced. This problem can be solved by running the entire linear program with Q_1, Q_2, and Q_3 as decision variables. This approach is equivalent to stream standards wherein the coordinator allocates the stream's assimilative capacity in an optimal manner. This is precisely the problem to be addressed here. The optimal solution is:

Minimize Z =	$\sum_{j=1}^{3} c_{1j}x_{1j} + (c+t_1)Q_1$	$+ \sum_{j=1}^{2} c_{2j}x_{2j} + (c+t_2)Q_2$	$+ \sum_{j=1}^{4} c_{3j}x_{3j} + (c+t_3)Q_3$	
Subject to				
Area 1's Problem	$\sum_{j=1}^{3} x_{1j} + Q_1$ x_{1j} x_{1j} Q_1	0	0	$= D_1$ $\le \bar{x}_{1j}$ for $j = 1,2,3$ ≥ 0 for $j = 1,2,3$ ≥ 0
Area 2's Problem	0	$\sum_{j=1}^{2} x_{2j} + Q_2$ x_{2j} x_{2j} Q_2	0	$= D_2$ $\le \bar{x}_{2j}$ for $j = 1,2$ ≥ 0 for $j = 1,2$ ≥ 0
Area 3's Problem	0	0	$\sum_{j=1}^{4} x_{3j} + Q_3$ x_{3j} x_{3j} Q_3	$= D_3$ $\le \bar{x}_{3j}$ for $j = 1,2,3,4$ ≥ 0 for $j = 1,2,3,4$
Coupling Constraints	Q_1	$+ \quad Q_2$	$+ \quad Q_3$	$\le \bar{W}$

Table 42.
OPTIMIZATION MODEL FOR THE THREE STUDY AREAS

$$\begin{array}{ll} Z_1 = \$1400 & Q_1 = 100 \\ Z_2 = 1020 & Q_2 = 270 \\ Z_3 = \underline{2650} & Q_3 = \underline{50} \\ \sum_{i=1}^{3} Z_i = \$5070:\ \pi = \$4 & \sum_{i=1}^{3} Q_i = 420 \end{array}$$

We see that the solution to the overall optimization problem reduced total costs from \$5,460 to \$5,070 by making more effective use of the available capacity. As can be seen by examining the solutions, the model took capacity from 1 and 3 and allotted it to 2 since his net savings (π_2) were higher. While this latter solution is the least costly from an overall point of view, if the costs are assigned as shown above, areas 1 and 3 are made worse off while area 2 is made significantly better off. Thus, areas 1 and 3 might reasonably object to such a solution. This is precisely the problem that has thwarted implementation of optimal programs, *i.e.*, while they are the least costly, they do not seem to be "fair" to everyone. We would like to have a procedure which is not only efficient, but also is equitable.

Perhaps one could use economic theory of demand to determine a better solution. The aggregate demand curve for the three areas is shown in Figure 36. Knowing the demand curve and the supply curve, then it is possible to determine the "market price" for the central facility. Referring to Figure 36, it is \$4/Q. Thus, according to economic theory, to achieve efficient resource allocation, a market price of \$4 per unit of storage should be used. Let π denote the market price (which is the same as the shadow price of central storage from the overall optimization model), then the assessment to each study area per unit of storage is $(\pi + c + t_i)$.

Economic theory does not address the distributional questions associated with using a market price concept. Thus, until recently, a very fundamental question was left unanswered as indicated below. First, if you actually charge the "market price" then a profit of 420π is realized. What do you do with the money? Traditionally, public services have been priced based on the cost of service. Thus, one could argue that such a price cannot be charged. Economic theory uses the notion of consumer surplus which is defined as the difference between an individual's willingness to pay and the actual assessment levied against him.

Let us determine the aggregate "consumer surplus" in this case if each area pays for off-site discharge it is allotted. As can be seen from this calculation,

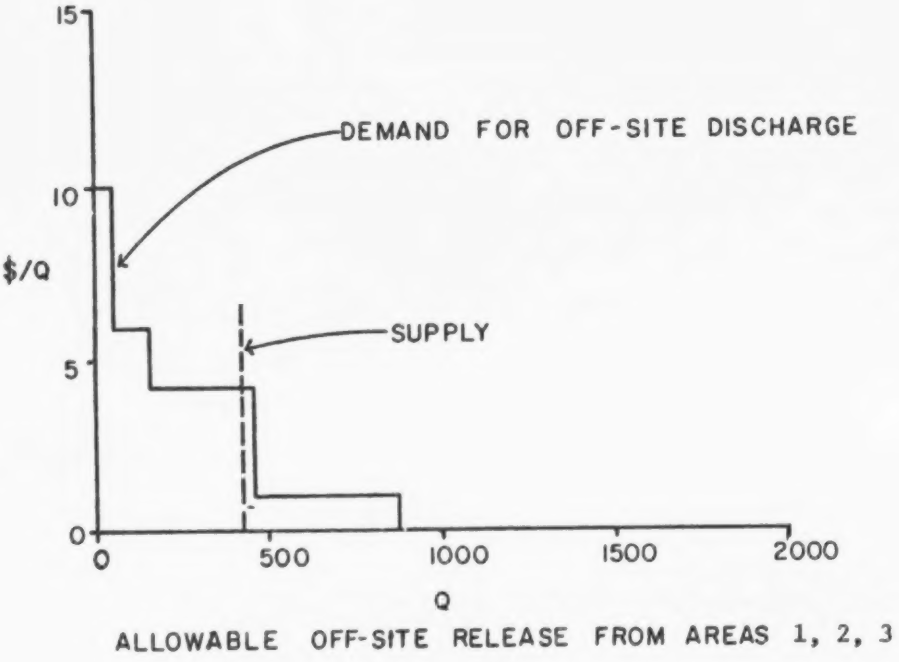


FIGURE 36.
Demand for Off-site Storage.

Area	Consumer Surplus if $(c + t_i)$ is Charged
1	6(100) = $ 600
2	4(270) = 1080
3	10(50) = 500
	$2180

the aggregate "consumer surplus" is $2180. In this example, the "consumer surplus" is the savings which accrue to each of the three areas because of the availability of 420 units of off-site disposal. This leads us to an important principle. Using this approach, it is possible to impute a value to the off-site facility based on the services it is performing which otherwise would have been required to be installed on-site. If the central facility is part of the natural system, say a swamp, flood plain, river, lake, or estuary, then a value of the natural system to these three areas is $2180 in this example. Thus, we have been able to derive a way of placing a value on the natural system based on the vital function it is providing for man! Interestingly, it is related to the notion of consumer surplus in economic theory.

This ability to place a value on the natural system is of vital importance in resource management. Traditionally, we have attempted to justify open space retention based on its recreation and aesthetic values. This has not provided convincing justification for preserving these areas. Using this technique one can prove, from a functional basis, that these areas are providing other valuable services for man.

Unfortunately, the problem is not yet solved. What happens if we attempt to implement the solution that charges only the actual cost but retains the optimal solution? If we do so, then areas 1 and 3 will probably object since area 2 derives most of the savings. To avoid this possibility one might charge the market price, $c + t_i + \pi$. If this is done, then the consumer surplus is as shown below.

Area	Consumer Surplus if $(c + t_i + \pi)$ is charged
1	2(100)= $ 200
2	0(270)= 0
3	6(50)= 300
Net Revenue to "Off-Site Facility"	4(420) 1680
	$2180

This solution has at least two problems: (1) what to do with the net revenue,and (2) the fact that any positive incentive to area 2 to participate has been eliminated since he is now indifferent between on-site control and off-site control.

The above problem is amenable to attack using N-person game theory. Luce and Raiffa (1957), Owen (1968), and Rapoport (1970) provide an introduction to the notions of N-person game theory. A key difference between two-person zero-sum game theory and cooperative N-person game theory is the idea of coalition formation and the allocation of associated transfer payments among the players. The balance of this section provides an axiomatic approach to the problem. A related procedure for cost-sharing has recently been proposed by Loehman and Whinston (1971).

Let $x\{i\}$ denote the cost assigned to the *ith* waste discharger if a cooperative arrangement is adopted and $c(N)$ denote the total cost for the coordinated solution. Then, the solutions which are admissible satisfy the following:

$$x\{i\} \geq 0 \text{ for all } i\varepsilon N \qquad (25)$$

$$\sum_{i\varepsilon N} x\{i\} = c(N) \qquad (26)$$

In game theory jargon all such solutions are group rational since the entire cost is partitioned among all of the waste dischargers. The question to be examined is: How should the costs of this central facility be apportioned among the three areas?

It is reasonable to require that

$$x\{i\} \leq c\{i\} \text{ for all } i\varepsilon N \qquad (27)$$

where $c\{i\}$ is the cost of the *ith* area if he acts independently . Thus, we simply require that the coordinated strategy cannot assess anyone more than their cost if they act independently. This is called the principle of individual rationality. The set of cost shares satisfying (25), (26), and (27) are called *imputations*. Furthermore, we may wish to extend (27) by requiring the following that for all subsets, S, of the master set, N:

$$\sum_{i\varepsilon S} x\{i\} \leq c(S) \text{ for all } S \subset N \qquad (28)$$

All solutions satisfying (25), (26), (27), and (28) belong to the *core* of the game.

Thus, the first major extension of existing procedures that is revealed using the game theoretic approach is that the notion of the core of an N-person game permits one to delimit more sharply the subset of solutions which are efficient and equitable. It does so by examining only those solutions which are rational for each individual or subset of individuals.

Returning to the example, there are $2^N - 1$ or 7 possible coalitions to be analyzed:

$$\{1\}\ \{2\}\ \{3\}$$

$$\{12\}\{13\}\{23\}$$

$$\{123\}$$

The next step in the analysis is to find $c(S)$ for the above combinations. $c(S)$ is called the *characteristic function* for coalition S and represents the total cost to that coalition. Usually, in game theory, it is assumed that the areas not in the coalition under consideration would form a directly competitive counter coalition which would lead to a minimax solution of a two-person game. Recently, S. Sorenson, (1972), a student of A. Charnes, generalized some of these notions of cooperative N-person game theory to the cases where alternative definitions of the characteristic function might be used. He describes four possible definitions which are listed below:

$c_1(S)$ = value to coalition if S is given preference over N - S

$c_2(S)$ = value of coalition to S if N - S is not present

$c_3(S)$ = value of coalition in a strictly competitive game between coalition S and N - S

$c_4(S)$ = value of coalition to S if N - S is given preference

As we shall see, alternative definitions can be used depending on how the problem is defined.

We would like to have a technique which permits us to obtain a unique solution to the problem which we can argue is equitable, or fair, to everyone.

The notion of the core reduces the set of feasible solutions considerably. However, there is still room for negotiation among the players as to how the costs are to be divided. Each area seeks to minimize its costs. Assume that the most recent entrant to a coalition pays these incremented costs. Therefore, the cost for the i*th* area is defined as

$$[c(S) - c(S-\{i\})] \text{ for all } S \subset N$$

Lacking prior knowledge regarding the likelihood of alternative coalition formation sequences, assume that all sequences may occur and are equilikely. Thus, there are 3! ways of forming the grand coalition as listed below:

123 213 312

132 231 321

Under these circumstances, we contend that a fair cost-sharing procedure is to assign to each area the expected value of the cost summed over the above coalition formation sequences. The general relationship is shown below. Let ϕ_i denote the cost assigned to the i*th* area where

$$\phi_i = \sum_{S \subseteq N} \frac{(S-1)!(n-S)!}{n!} [c(S) - c(S - \{i\})]$$

The assigned cost, ϕ_i, is called the Shapley value for N-person games (Shapley, 1953). Note that,

$$\sum_{i \in N} \phi_i \; c(N)$$

so that the entire cost is apportioned. The Shapley value is a very useful solution notion. Unfortunately, it may happen that the Shapley value can fall outside the core of the game (Sorenson, 1972). We want to avoid such cases.

We have examined several problems in cost-sharing and resource allocation. It works out that different definitions of the characteristic function are appropriate depending on how the problem is defined. The easier case is the cost-sharing problem where ownership has been established and the decision needs to be made whether to control on-site only or to form a coalition with others. The more difficult case is to determine a solution when ownership of the central facilities is unspecified. These two cases are considered below.

From a computational point of view, $c_1(S)$ can be determined by solving a partition of the overall optimization problem. This procedure will be used in solving the example for various cases. Recall that the overall objective function is to minimize the costs for all three areas. Assume there are n goals over which we want to optimize, *i.e.*,

$$f_1(x),\ f_2(x)\ .\ .\ .\ f_n(x)$$

In our case the n separate goals represent the goals of all coalition combinations. Then the values of the characteristic function for a given coalition is found by determining the optimal solution to the linear program for the specified objective function. The following cases are considered.

Case 1: Ownership of off-site control is specified. Each area "owns" 140 units of the central facility which they may use or sell to other areas.

In this case each coalition seeks to minimize its costs given a specified allocation of the off-site discharge $\overline{W}$. Let $[f_i(x):Q_i \leq W']$ denote the objective function for the ith coalition with a specified allocation $\overline{W}'$ of the total of $\overline{W}$ units. The results of this calculation are shown in Table 43. Note that the coalition, $\overline{13}$ is inessential, *i.e.*, $c(\overline{13}) = c(1) + c(3)$.

The core of this game is delimited by

$$\begin{array}{lllll}
x_1 & & & \leq & 1360 \\
& x_2 & & \leq & 1540 \\
& & x_3 & \leq & 2560 \\
x_1 & + x_2 & & \leq & 2780 \\
x_1 & & + x_3 & \leq & 3920 \\
& x_2 & + x_3 & \leq & 3830 \\
x_1 & + x_2 & + x_3 & = & 5070
\end{array}$$

The Shapley values are

$$\begin{aligned}
\phi_1 = {} & \frac{1}{3}(1360) + \frac{1}{6}(2780 - 1540) + \frac{1}{6}(3920 - 2560) \\
& + \frac{1}{3}(5070 - 3830)
\end{aligned}$$

Table 43. OPTIMAL SOLUTION WITH OWNERSHIP
OF OFF-SITE FACILITY PRESPECIFIED

Coalition Cost	Objective function	Resource allocation: $\overline{W}'$	Optimal solution
$c(1)$	$f_1(x) = Z_1$	140	\$1360
$c(2)$	$f_2(x) = Z_2$	140	1540
$c(3)$	$f_3(x) = Z_3$	140	2560
$c(\overline{12})$	$f_4(x) = Z_{12}$	280	2780
$c(\overline{13})$	$f_5(x) = Z_{13}$	280	3920
$c(\overline{23})$	$f_6(x) = Z_{23}$	280	3830
$c(\overline{123})$	$f_7(x) = Z_{123}$	420	5070

$$\phi_1 = \$1300$$

$$\phi_2 = \frac{1}{3}(1540) + \frac{1}{6}(2780 - 1360) + \frac{1}{6}(3830 - 2560) + \frac{1}{3}(5070 - 3920)$$

$$\phi_2 = \$1345$$

$$\phi_3 = \frac{1}{3}(2560) + \frac{1}{6}(3920 - 1360) + \frac{1}{6}(3830 - 1540) + \frac{1}{3}(5070 - 2780)$$

$$\phi_3 = \$2425$$

Note that $\sum_{i=1}^{3} \phi_i = \$5070$ and that the Shapley value does fall within the core of the game. In this case the consumer surplus or savings from having ownership of 140 units is the difference between the cost of on-site control and the Shapley value or \$700, \$755, and \$725 for areas 1, 2, and 3, respectively. This is a much more uniform distribution of savings than existed before.

Case 2: Ownership of off-site control is unspecified. Coordinators own the central facility and allocate it in an efficient and equitable manner.

It has been demonstrated earlier that the optimal solution allocates 100 units to area 1, 270 units to area 2, and 50 units to area 3. The unit costs of providing this central facility are \$4, \$3, and \$5, respectively. At this cost, a larger savings accrues to area 2 than to either areas 1 and 3. Furthermore, areas 1 and 3 would be willing to pay more than \$4 to obtain additional units from 2. Thus, they might feel that the solution is unfair.

Let us explore the possibility of using cooperative N-person game theory to solve this problem. In Case 1, ownership of the central facility was prespecified. Thus, the values of c(S) were unambiguously defined. This corresponded to the second definition of the characteristic function, $c_2(S)$, wherein the other coalition (N - S) could be viewed as not being present.

Let us examine definition one wherein $c_1(S)$ is defined as the value if S is given preference over N - S. In this case S is allotted as much of the off-site facility as he desires. The resulting values of the characteristic function are shown below:

$$c_1(1) = \$1200;\ c_1(2) = \$900;\ c_1(3) = \$2450$$

$$c_1(12) = \$2280;\ c_1(13) = \$3780;\ c_1(23) = \$3480$$

$$c_1(123) = \$5070$$

Inspection reveals that this game is not subadditive since coalitions are never advantageous, *e.g.*, $c(\overline{12}) > c(1) + c(2)$. Thus, this definition of the characteristics function would not result in cooperation in this case since coalitions never leave the players better off.

Let us try the opposite definition wherein N - S is given preference over S. The resulting characteristic function values are shown below:

$$c_4(1) = \$2000;\ c_4(2) = \$2100;\ c_4(3) = \$3150$$

$$c_4(12) = \$3220;\ c_4(13) = \$4230;\ c_4(23) = \$4470$$

$$c_4(123) = \$5070$$

Unlike the previous situation this game is subadditive for all coalitions. Thus, there is a basis for cooperation.

Indeed one can generalize that in this example coalitions are never advantageous using $c_1(S)$ and are never disadvantageous using $c_4(S)$. The reason is simple. In the former case, the individual area is allotted up to the total available storage, $\overline{W}$, in determining c(1), c(2), and c(3). However, as coalitions form he is forced to share $\overline{W}$ with other areas. Thus, he is never made better off if an additional area joins the coalition since he cannot (due to the nonnegativity restrictions on Q_i) attain more of the central facility than he had before. At best, he is indifferent. On the other hand, using $c_4(S)$, the area will always consider coalitions since they may thereby be able to bid for a larger amount of the capacity of the central facility. Note that $c_1(123) = c_4(123)$ so that the value of the grand coalition is the same. The major difference is in the value of the subcoalitions. This analysis illustrates the importance of a fundamental aspect of resource allocation--the question of ownership. In Case 1 ownership was specified *a priori*. This case leads to a pervasively cooperative situation since larger coalitions can never be harmful. Each area has the option of using its allotted share of the off-site facility or selling it to others. Similarly, in the situation where (N - S) is given preference, *i.e.*, $c_4(S)$, a cooperative situation results. However, in the case where S is given priority, a pervasively competitive situation arises.

This discussion can be summarized by introducing the notion of *security level* which is defined as the value the player can assure himself if he acts independently. We see that the security level depends on how the characteristic function is defined. For the case where ownership is not specified *a priori*, only definition 4 of the characteristic function guarantees a cooperative solution. Definitions 1 and 3 lead to strict competition and definition 2 is irrelevant since one cannot assume the other N - S players are absent in the case where ownership of the common resource is unspecified. Thus, we feel that definition 4 provides a reasonable basis for an equitable cost sharing arrangement. The Shapley values for this case are:

$$\phi_1 = \frac{1}{3}(2000) + \frac{1}{6}(3220 - 2100) + \frac{1}{6}(4230 - 3150) + \frac{1}{3}(5070 - 4470)$$

$$\phi_1 = \$1233$$

$$\phi_2 = \frac{1}{3}(2100) + \frac{1}{6}(3220 - 2000) + \frac{1}{6}(4470 - 3150) + \frac{1}{3}(5070 - 4230)$$

$$\phi_2 = \$1400$$

$$\phi_3 = \frac{1}{3}(3150) + \frac{1}{6}(4230 - 2000) + \frac{1}{6}(4470 - 2100) + \frac{1}{3}(5070 - 3220)$$

$$\phi_3 = \$2437$$

Note the $\sum_{i=1}^{3} \phi_i = \$5070$ and the solution is in the core.

The consumer surplus for this case is $767, $700, and $713, respectively. One could argue that, as long as the calculated consumer surpluses are almost the same, wouldn't it be fairer to simply equalize the consumer surplus so that each area saved an equal amount--about $727 per study area in this case? Table 44 shows the charges to each study area for the above two cases.

In actual situationsone might decide that every area should pay at least a minimum amount or impose other restrictions. It is straightforward to formulate this problem using goal programming wherein one can minimize the sum of the positive and negative deviations of the excesses relative to a uniform savings solution (see Charnes and Cooper (1961) for a description of goal programming).

Table 44. CHARGES FOR CENTRAL FACILITY

		Total cost				
Area	Allocation of storage	Shapley	equal savings	On-site cost	Charge Shapley	$\$/Q_i$ equal savings
1	100	\$1233	\$1273	\$1000	\$2.33	\$2.73
2	270	1400	1373	210	4.41	4.32
3	50	2437	2424	2400	0.74	0.48
Totals	420	\$5070	\$5070	\$3610	----	----

The problem of efficient and equitable urban stormwater quality management has been addressed using recently developed notions from cooperative N-person game theory. Numerous investigators have demonstrated the value of optimization techniques for solving the problem of determining the pollution control strategy which is in the best overall interest. Unfortunately, such procedures often prescribed solutions which did not seem fair to everyone. Whether or not the solution was fair depends on the cost sharing arrangements which in turn depend on who is entitled to off-site disposal of pollution.

Using a simple example, it has been shown how the optimal solution is determined using linear programming or a simple market price determination. Included in this analysis is an explicit procedure for placing a value on off-site disposal which is based on savings in alternative on-site control costs. This procedure provides a way of quantifying a value for the natural system which is the recipient of these wastes. Difficulties in achieving an equitable solution were outlined.

Next, introductory concepts from cooperative N-person game theory were presented. This section indicated how it is possible to devise management strategies which retain the efficient solution and permit an equitable cost sharing arrangement to be made. Then two cases were examined. In the first case, ownership of the off-site facility was prespecified. Polluters in this situation could utilize their right or exchange it with others. In the second case, ownership was not specified. Depending on how one defined the characteristic function, a competitive or cooperative situation would result. The results of the analysis provide a way of solving this problem.

SECTION VII

HYDRAULIC DESIGN BY THE SWMM

A. METHODS

Current practice in hydraulic design of storm and combined sewer systems still relies heavily upon the rational method in which peak flow rates are computed and summed at sewer junctions on the basis of assumed runoff coefficients, times of concentration and full-flow pipe capacities (ASCE, 1969). It has been shown that peak flow rates in excess of those predicted on the basis of steady, uniform flow can be accommodated by conduits when flow dynamics are included, as is done in the SWMM (Henderson, 1966). Consequently, a design utilizing the SWMM can be expected to produce a more efficient (smaller) conduit size than would a rational method formulation.

A further advantage accrues through the use of the RUNOFF portion of the SWMM wherein a dynamic routine is used to generate the inlet hydrographs. Temporal and spatial variations of surface runoff are fully represented.

The Road Research Laboratory (RRL) method has been successfully employed in Britain to overcome several of the deficiencies of the rational method. However, its surface and sewer routing procedures are less sophisticated than those of the SWMM, and it has been shown in an EPA study to become less accurate as the percent imperviousness decreases (Stall and Terstriep, 1972). This is due to the fact that the RRL model computes runoff only from directly connected impervious areas.

A hydraulic design capability has been installed in the SWMM. Basically, the routine checks for surcharge conditions and, if present, enlarges the downstream conduit by standard amounts until capacity exists to accept the flow. In its present form it is best suited for redesign of existing systems inasmuch as the sewer layout, slopes and first estimates of sizes must be supplied *a priori*. Revisions are intended to allow the possibility of upstream storage as an alternative to increased conduit capacity. Eventually the program may be enlarged to allow possible changes in slope, if constraints on invert elevations are supplied. Finally, minimization of excavation is also a possibility if surface elevations are also supplied.

The "design storm" must be 1) chosen from the historical record; or 2) synthesized from intensity-duration curves and percentage-mass curves as done by Stall and Terstriep (1972); or 3) synthesized by other means (Sorman and Wallace, 1972). However, it should be remembered that the return period of the runoff will not necessarily be the same as that of the rainfall due to differences in antecedent conditions and other factors affecting the catchment from storm to storm.

B. EXAMPLE

The design method is shown for Storm Study Number 6 on Stevens Avenue. Initial conduit dimensions are shown in Table 45. Elements 137 and 143 have been replaced by horseshoe and rectangular shapes, respectively, to demonstrate the methodology (see Figure 17 for sewer layout). Currently, if additional capacity is required, diameters of circular conduits are increased by three inches if presently less than three feet or six inches if greater. Rectangular conduits are increased in width by six inches, under the assumption that system constraints are more likely to limit vertical dimensions than lateral. Non-circular or non-rectangular conduits are converted to equivalent (equal area) circular conduits of standard sizes and treated as such in subsequent time steps.

Table 46 illustrates the output at each time step. Dimensions are increased until surcharging no longer exists at a given time step, and flow routing proceeds under those conditions. The ratio of required flow capacity to current full-flow capacity is also indicated so that the importance of the surcharging can be evaluated. If the ratio is near 1.0 alterations are perhaps unnecessary. For instance, conduit number 168 appears to be definitely undersized whereas an enlargement of conduit number 147 may be only of marginal necessity. Table 47 illustrates the final conduit dimensions. Altered values are indicated.

An obvious trade-off exists between lack of surcharging with increased conduit sizes versus lower peak flow rates at the outlet when upstream surcharging is allowed. This is illustrated in Figure 37 in which the peak outlet flow rate is increased by 75 cfs (2.2 m^3/sec) when conduit sizes are increased. Surcharging, or other more acceptable upstream storage alternatives, may provide valuable relief in reduction of peak outlet flow rates.

Finally, the interrelationship between surcharging locations and points of high-inflow volume as indicated by the monitoring routine (Section IV-G) should be noted. Many of the surcharge locations as indicated on Table 46 correspond to the inlet locations with high-total inflow shown on Table 21 (although the examples are for two different storms). The volumes indicated on either Table 46 or Table 21 could serve as first estimates of capacities required for upstream storage facilities.

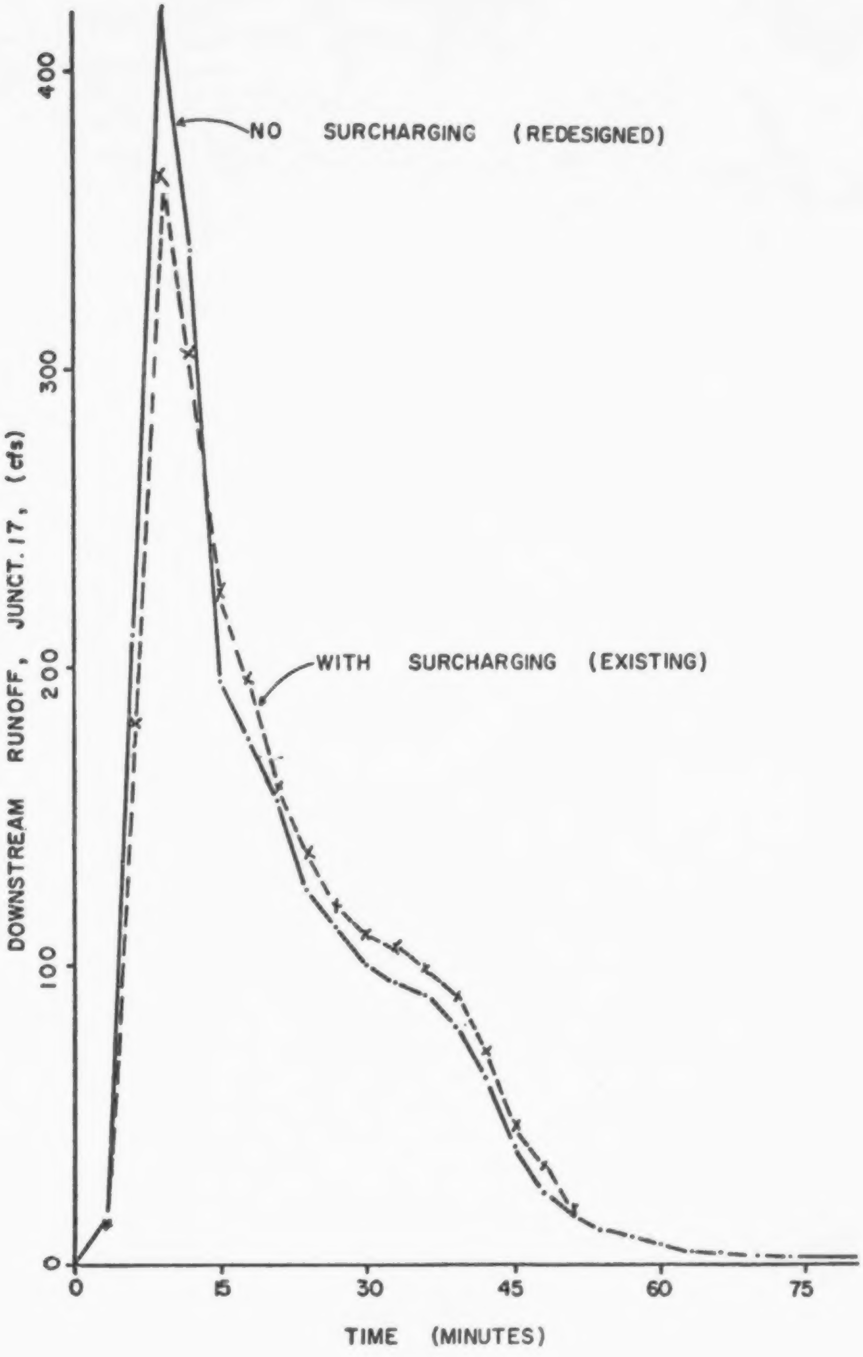


FIGURE 37.
Effect of Surcharging and Conduit Alteration on Stevens Avenue Overflow. Study 1, Storm 6.

```
      10 YEAR STORM OF AUGUST 3, 1971 DURATION 1 HOUR (STORM #6)

          NUMBER OF ELEMENTS=   95
          NUMBER OF TIME INT=   78
               TIME INTERVAL=  180.0 SECONDS.

 ELEMENT PARAMETERS
  EXT. TYPE  DESCRIPTION        SLOPE   DISTANCE  MANNING    GEOM1  GEOM2  GEOM3  NUMBER   AFULL    QFULL     QMAX     SUPER-CRITICAL
  ELE.                         (FT/FT)    (FT)   ROUGHNESS   (FT)   (FT)   (FT)     OF    (SQ.FT)   (CFS)     (CFS)    FLOW WHEN LESS
  NUM.                                                                           BARRELS                              THAN 95% FULL?
  118    1   CIRCULAR SHAPED  0.04050   201.00    0.0130    5.000  0.0    0.0      1.0    19.635   525.541   567.504      YES
  119    1   CIRCULAR SHAPED  0.03956   159.00    0.0130    5.000  0.0    0.0      1.0    19.635   519.406   560.959      YES
  120    1   CIRCULAR SHAPED  0.03504   274.00    0.0130    5.000  0.0    0.0      1.0    19.635   488.834   527.840      YES
  128    1   CIRCULAR SHAPED  0.03829   298.00    0.0130    5.000  0.0    0.0      1.0    19.635   511.001   551.801      YES
  129    1   CIRCULAR SHAPED  0.02859   270.00    0.0130    5.000  0.0    0.0      1.0    19.635   441.556   476.801      YES
  130    1   CIRCULAR SHAPED  0.04672   600.00    0.0130    5.000  0.0    0.0      1.0    19.635   564.457   609.513      YES
  131    1   CIRCULAR SHAPED  0.01848   270.00    0.0130    3.500  0.0    0.0      1.0     9.621   129.505   139.845      NO
  135    1   CIRCULAR SHAPED  0.02000   448.00    0.0130    3.500  0.0    0.0      1.0     9.621   142.667   154.080      YES
  137    4   HORSE SHOE       0.04556   270.00    0.0130    2.000  0.0    0.0      1.0     3.317    51.640    55.616      YES
  138    1   CIRCULAR SHAPED  0.04855   420.00    0.0130    0.670  0.0    0.0      1.0     0.353     2.706     2.922      YES
  140    1   CIRCULAR SHAPED  0.03473   150.00    0.0130    2.000  0.0    0.0      1.0     3.142    42.273    45.654      YES
  143    2   RECTANGULAR      0.02305   400.00    0.0130    2.000  1.500  0.0      1.0     3.000    29.674    33.616      YES
  146    1   CIRCULAR SHAPED  0.04952   484.00    0.0130    2.000  0.0    0.0      1.0     3.142    50.477    54.515      YES
  147    1   CIRCULAR SHAPED  0.01358   446.00    0.0130    1.250  0.0    0.0      1.0     1.227     7.548     8.152      NO
  149    1   CIRCULAR SHAPED  0.02768   250.00    0.0130    1.000  0.0    0.0      1.0     0.785     5.943     6.419      YES
  150    1   CIRCULAR SHAPED  0.01765   566.00    0.0130    5.000  0.0    0.0      1.0    19.635   346.938   374.693      YES
  152    1   CIRCULAR SHAPED  0.03476   420.00    0.0130    3.000  0.0    0.0      1.0     7.069   124.687   134.662      YES
  153    1   CIRCULAR SHAPED  0.01725   290.00    0.0130    3.000  0.0    0.0      1.0     7.069    87.837    94.846      NO
  154    1   CIRCULAR SHAPED  0.03871   388.00    0.0130    3.000  0.0    0.0      1.0     7.069   131.581   142.100      YES
  155    1   CIRCULAR SHAPED  0.02353   312.00    0.0130    2.500  0.0    0.0      1.0     4.909    63.087    68.134      YES
  157    1   CIRCULAR SHAPED  0.03161   280.00    0.0130    2.000  0.0    0.0      1.0     3.142    40.329    43.555      YES
  158    1   CIRCULAR SHAPED  0.06258   200.00    0.0130    1.000  0.0    0.0      1.0     0.785     8.937     9.652      YES
  159    1   CIRCULAR SHAPED  0.06115   358.00    0.0130    1.000  0.0    0.0      1.0     0.785     8.834     9.541      YES
  160    1   CIRCULAR SHAPED  0.03094   256.00    0.0130    2.000  0.0    0.0      1.0     3.142    39.899    43.091      YES
  161    1   CIRCULAR SHAPED  0.03164   250.00    0.0130    2.000  0.0    0.0      1.0     3.142    40.348    43.576      YES
  162    1   CIRCULAR SHAPED  0.05332   515.00    0.0130    2.000  0.0    0.0      1.0     3.142    52.378    56.560      YES
  163    1   CIRCULAR SHAPED  0.01647   498.00    0.0130    2.000  0.0    0.0      1.0     3.142    29.111    31.440      NO
  164    1   CIRCULAR SHAPED  0.01306    85.00    0.0130    1.500  0.0    0.0      1.0     1.767    12.037    13.000      NO
  165    1   CIRCULAR SHAPED  0.04433   240.00    0.0130    1.500  0.0    0.0      1.0     1.767    22.176    23.950      YES
  166    1   CIRCULAR SHAPED  0.02430   367.00    0.0130    0.670  0.0    0.0      1.0     0.353     1.914     2.067      NO
  167    1   CIRCULAR SHAPED  0.01125   255.00    0.0130    4.000  0.0    0.0      1.0    12.566   152.767   164.988      NO
  168    1   CIRCULAR SHAPED  0.04662   420.00    0.0130    1.000  0.0    0.0      1.0     0.785     7.713     8.330      YES
  169    1   CIRCULAR SHAPED  0.00784   190.00    0.0130    0.830  0.0    0.0      1.0     0.541     1.925     2.078      NO
  171    1   CIRCULAR SHAPED  0.04421   271.00    0.0130    4.000  0.0    0.0      1.0    12.566   302.840   327.007      YES
  175    1   CIRCULAR SHAPED  0.05564   250.00    0.0130    4.000  0.0    0.0      1.0    12.566   339.740   366.858      YES
  176    1   CIRCULAR SHAPED  0.05496   280.00    0.0130    1.500  0.0    0.0      1.0     1.767    24.692    26.668      YES
  179    1   CIRCULAR SHAPED  0.02245   579.00    0.0130    4.000  0.0    0.0      1.0    12.566   215.805   233.069      YES
  180    1   CIRCULAR SHAPED  0.02333   249.00    0.0130    3.000  0.0    0.0      1.0     7.069   102.150   110.322      YES
  181    1   CIRCULAR SHAPED  0.02690   171.00    0.0130    3.000  0.0    0.0      1.0     7.069   109.688   118.463      YES
  182    1   CIRCULAR SHAPED  0.02172   238.00    0.0130    3.000  0.0    0.0      1.0     7.069    98.563   106.428      YES
  183    1   CIRCULAR SHAPED  0.04098   255.00    0.0130    1.000  0.0    0.0      1.0     0.785     7.232     7.810      YES
  189    1   CIRCULAR SHAPED  0.03735   340.00    0.0130    1.500  0.0    0.0      1.0     1.767    20.355    21.980      YES
  191    1   CIRCULAR SHAPED  0.02533   210.00    0.0130    1.500  0.0    0.0      1.0     1.767    16.763    18.104      YES
  193    1   CIRCULAR SHAPED  0.02546   205.00    0.0130    1.000  0.0    0.0      1.0     0.785     5.700     6.156      NO
  197    1   CIRCULAR SHAPED  0.05306   422.00    0.0130    2.000  0.0    0.0      1.0     3.142    52.250    56.430      YES
  198    1   CIRCULAR SHAPED  0.01059   340.00    0.0130    1.000  0.0    0.0      1.0     0.785     3.676     3.970      NO
  199    1   CIRCULAR SHAPED  0.02000   250.00    0.0130    1.500  0.0    0.0      1.0     1.767    14.895    16.087      NO

 EPSILON = 0.000100      NO. OF ITERATIONS IN ROUTING ROUINE =    4
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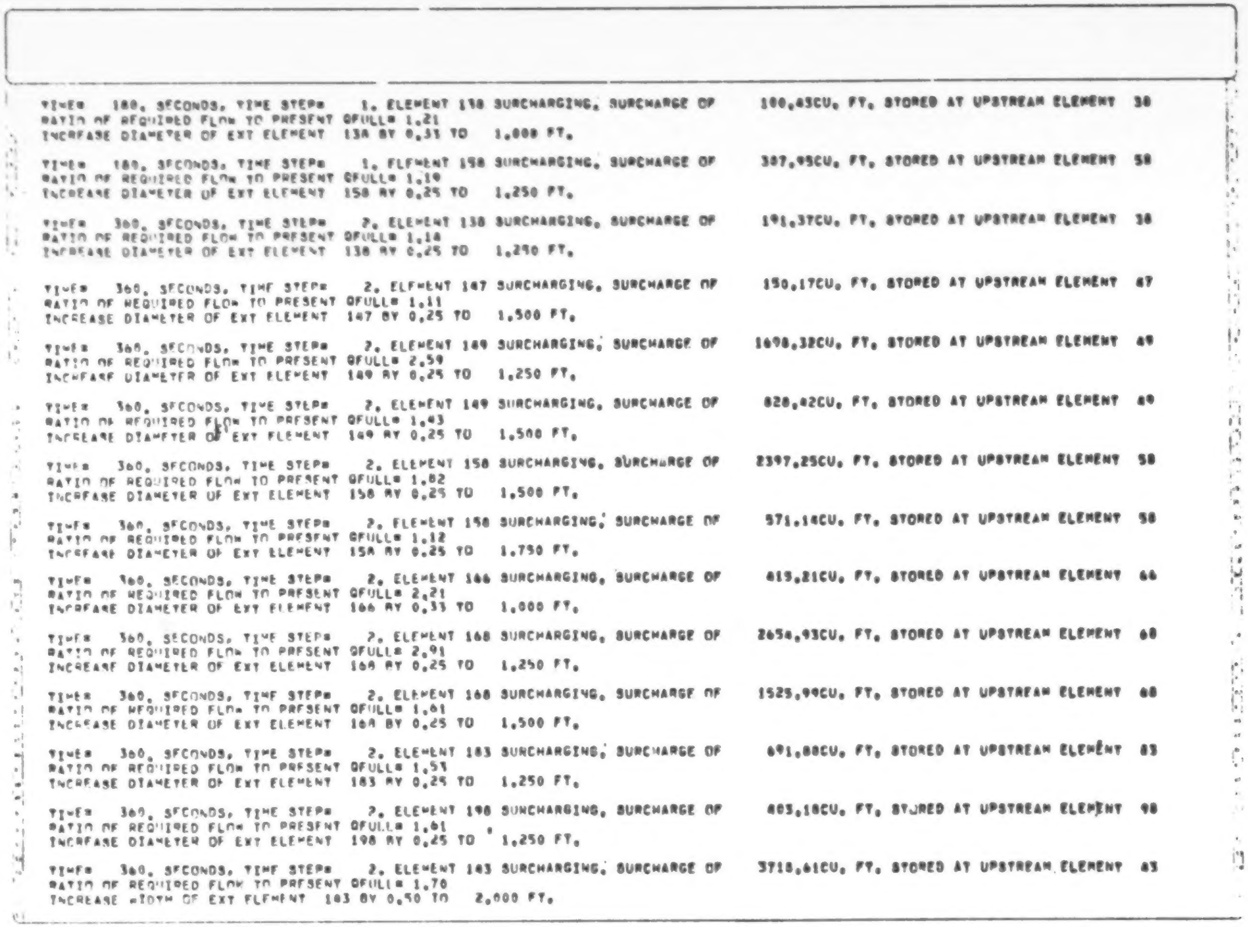
Table 45.
INITIAL DIMENSIONS AND CONDUIT CAPACITIES.



```
TIME=   180. SECONDS, TIME STEP=    1. ELEMENT 138 SURCHARGING, SURCHARGE OF     180.43CU. FT. STORED AT UPSTREAM ELEMENT  38
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.21
INCREASE DIAMETER OF EXT ELEMENT  138 BY 0.33 TO   1.000 FT.

TIME=   180. SECONDS, TIME STEP=    1. ELEMENT 158 SURCHARGING, SURCHARGE OF     307.95CU. FT. STORED AT UPSTREAM ELEMENT  58
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.19
INCREASE DIAMETER OF EXT ELEMENT  158 BY 0.25 TO   1.250 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 138 SURCHARGING, SURCHARGE OF     191.37CU. FT. STORED AT UPSTREAM ELEMENT  38
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.14
INCREASE DIAMETER OF EXT ELEMENT  138 BY 0.25 TO   1.250 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 147 SURCHARGING, SURCHARGE OF     150.17CU. FT. STORED AT UPSTREAM ELEMENT  47
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.11
INCREASE DIAMETER OF EXT ELEMENT  147 BY 0.25 TO   1.500 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 149 SURCHARGING, SURCHARGE OF    1698.32CU. FT. STORED AT UPSTREAM ELEMENT  49
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 2.59
INCREASE DIAMETER OF EXT ELEMENT  149 BY 0.25 TO   1.250 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 149 SURCHARGING, SURCHARGE OF     828.42CU. FT. STORED AT UPSTREAM ELEMENT  49
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.43
INCREASE DIAMETER OF EXT ELEMENT  149 BY 0.25 TO   1.500 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 158 SURCHARGING, SURCHARGE OF    2397.25CU. FT. STORED AT UPSTREAM ELEMENT  58
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.82
INCREASE DIAMETER OF EXT ELEMENT  158 BY 0.25 TO   1.500 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 158 SURCHARGING, SURCHARGE OF     571.14CU. FT. STORED AT UPSTREAM ELEMENT  58
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.12
INCREASE DIAMETER OF EXT ELEMENT  158 BY 0.25 TO   1.750 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 166 SURCHARGING, SURCHARGE OF     815.21CU. FT. STORED AT UPSTREAM ELEMENT  66
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 2.21
INCREASE DIAMETER OF EXT ELEMENT  166 BY 0.33 TO   1.000 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 168 SURCHARGING, SURCHARGE OF    2654.93CU. FT. STORED AT UPSTREAM ELEMENT  68
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 2.91
INCREASE DIAMETER OF EXT ELEMENT  168 BY 0.25 TO   1.250 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 168 SURCHARGING, SURCHARGE OF    1525.99CU. FT. STORED AT UPSTREAM ELEMENT  68
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.61
INCREASE DIAMETER OF EXT ELEMENT  168 BY 0.25 TO   1.500 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 183 SURCHARGING, SURCHARGE OF     691.88CU. FT. STORED AT UPSTREAM ELEMENT  83
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.53
INCREASE DIAMETER OF EXT ELEMENT  183 BY 0.25 TO   1.250 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 198 SURCHARGING, SURCHARGE OF     403.18CU. FT. STORED AT UPSTREAM ELEMENT  98
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.61
INCREASE DIAMETER OF EXT ELEMENT  198 BY 0.25 TO   1.250 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 183 SURCHARGING, SURCHARGE OF    3718.61CU. FT. STORED AT UPSTREAM ELEMENT  83
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.70
INCREASE WIDTH OF EXT ELEMENT  183 BY 0.50 TO   2.000 FT.
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Table 46.
HISTORY OF SURCHARGING AND CONDUIT ALTERATION DURING STORM.

```
TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 140 SURCHARGING, SURCHARGE OF    2958.92CU. FT. STORED AT UPSTREAM ELEMENT  40
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.39
INCREASE DIAMETER OF EXT ELEMENT  140 BY 0.25 TO   2.250 FT.
TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 137 SURCHARGING, SURCHARGE OF    5092.32CU. FT. STORED AT UPSTREAM ELEMENT  37
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.55
REPLACE EXT ELEMENT  137 BY CIRCULAR CONDUIT OF DIAMETER   2.250 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 137 SURCHARGING, SURCHARGE OF    2456.58CU. FT. STORED AT UPSTREAM ELEMENT  37
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.21
INCREASE DIAMETER OF EXT ELEMENT  137 BY 0.25 TO   2.500 FT.

TIME=   360. SECONDS, TIME STEP=    2. ELEMENT 153 SURCHARGING, SURCHARGE OF    5446.76CU. FT. STORED AT UPSTREAM ELEMENT  53
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.34
INCREASE DIAMETER OF EXT ELEMENT  153 BY 0.50 TO   3.500 FT.
```

Table 46 (cont).

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FINAL CONDUIT DIMENSIONS. ASTERISK INDICATES ALTERED CONDUITS.

FLEMENT PARAMETERS
EXT. TYPE  DESCRIPTION          SLOPE   DISTANCE  MANNING    GEOM1  GEOM2  GEOM3  NUMBER   AFULL    QFULL     QMAX    SUPER-CRITICAL
ELE.                           (FT/FT)    (FT)   ROUGHNESS   (FT)   (FT)   (FT)     OF    (SQ.FT)   (CFS)     (CFS)   FLOW WHEN LESS
NUM.                                                                             BARRELS                             THAN 95% FULL?
 118    1    CIRCULAR SHAPED  0.04050   201.00    0.0130    5.000  0.0    0.0      1.0   19.635   525.541   567.586     YES
 119    1    CIRCULAR SHAPED  0.03956   159.00    0.0130    5.000  0.0    0.0      1.0   19.635   519.400   560.959     YES
 120    1    CIRCULAR SHAPED  0.03504   274.00    0.0130    5.000  0.0    0.0      1.0   19.635   488.838   527.940     YES
 128    1    CIRCULAR SHAPED  0.03829   298.00    0.0130    5.000  0.0    0.0      1.0   19.635   511.001   551.881     YES
 129    1    CIRCULAR SHAPED  0.02859   270.00    0.0130    5.000  0.0    0.0      1.0   19.635   441.556   476.881     YES
 130    1    CIRCULAR SHAPED  0.04672   600.00    0.0130    5.000  0.0    0.0      1.0   19.635   564.057   609.613     YES
 131    1    CIRCULAR SHAPED  0.01648   270.00    0.0130    3.500  0.0    0.0      1.0    9.621   129.505   139.865     NO
 135    1    CIRCULAR SHAPED  0.02000   448.00    0.0130    3.500  0.0    0.0      1.0    9.621   142.667   154.080     YES
 137    1   *CIRCULAR SHAPED  0.04556   270.00    0.0130    2.500  0.0    0.0      1.0    4.909    87.786    94.809     YES
 139    1   *CIRCULAR SHAPED  0.04855   428.00    0.0130    1.250  0.0    0.0      1.0    1.227    14.272    15.414     YES
 142    1   *CIRCULAR SHAPED  0.03473   150.00    0.0130    2.250  0.0    0.0      1.0    3.976    57.871    62.501     YES
 143    2   *RECTANGULAR      0.02305   400.00    0.0130    2.000  2.000  0.0      1.0    4.000    43.848    51.174     YES
 146    1    CIRCULAR SHAPED  0.04952   484.00    0.0130    2.000  0.0    0.0      1.0    3.142    50.477    54.515     YES
 147    1   *CIRCULAR SHAPED  0.01358   466.00    0.0130    1.500  0.0    0.0      1.0    1.767    12.278    13.256     NO
 149    1   *CIRCULAR SHAPED  0.02768   250.00    0.0130    1.500  0.0    0.0      1.0    1.767    17.523    18.925     YES
 150    1    CIRCULAR SHAPED  0.01765   566.00    0.0130    5.000  0.0    0.0      1.0   19.635   346.938   374.693     YES
 152    1    CIRCULAR SHAPED  0.03476   420.00    0.0130    3.000  0.0    0.0      1.0    7.069   124.687   134.662     YES
 153    1   *CIRCULAR SHAPED  0.01725   200.00    0.0130    3.500  0.0    0.0      1.0    9.621   132.496   143.095     NO
 154    1    CIRCULAR SHAPED  0.03871   388.00    0.0130    3.000  0.0    0.0      1.0    7.069   131.581   142.108     YES
 155    1    CIRCULAR SHAPED  0.02353   312.00    0.0130    2.500  0.0    0.0      1.0    4.909    63.087    68.134     YES
 157    1    CIRCULAR SHAPED  0.03161   280.00    0.0130    2.000  0.0    0.0      1.0    3.142    40.329    43.555     YES
 158    1   *CIRCULAR SHAPED  0.06258   240.00    0.0130    1.750  0.0    0.0      1.0    2.405    39.745    42.924     YES
 159    1    CIRCULAR SHAPED  0.06115   358.00    0.0130    1.000  0.0    0.0      1.0    0.785     8.834     9.541     YES
 160    1    CIRCULAR SHAPED  0.03094   256.00    0.0130    2.000  0.0    0.0      1.0    3.142    39.899    43.091     YES
 161    1    CIRCULAR SHAPED  0.03144   250.00    0.0130    2.000  0.0    0.0      1.0    3.142    40.348    43.576     YES
 162    1    CIRCULAR SHAPED  0.05332   515.00    0.0130    2.000  0.0    0.0      1.0    3.142    52.378    56.568     YES
 163    1    CIRCULAR SHAPED  0.01647   498.00    0.0130    2.000  0.0    0.0      1.0    3.142    29.111    31.440     NO
 164    1    CIRCULAR SHAPED  0.01306    85.00    0.0130    1.500  0.0    0.0      1.0    1.767    12.037    13.000     NO
 165    1    CIRCULAR SHAPED  0.04433   240.00    0.0130    1.500  0.0    0.0      1.0    1.767    22.176    23.950     YES
 166    1   *CIRCULAR SHAPED  0.02430   367.00    0.0130    1.000  0.0    0.0      1.0    0.785     5.569     6.014     NO
 167    1    CIRCULAR SHAPED  0.01125   255.00    0.0130    4.000  0.0    0.0      1.0   12.566   152.767   164.988     NO
 168    1   *CIRCULAR SHAPED  0.04662   420.00    0.0130    1.500  0.0    0.0      1.0    1.767    22.742    24.561     YES
 169    1    CIRCULAR SHAPED  0.00784   190.00    0.0130    0.830  0.0    0.0      1.0    0.541     1.925     2.078     NO
 171    1    CIRCULAR SHAPED  0.04421   271.00    0.0130    4.000  0.0    0.0      1.0   12.566   302.840   327.067     YES
 175    1    CIRCULAR SHAPED  0.05558   250.00    0.0130    4.000  0.0    0.0      1.0   12.566   339.740   366.918     YES
 176    1    CIRCULAR SHAPED  0.05496   210.00    0.0130    1.500  0.0    0.0      1.0    1.767    24.692    26.668     YES
 179    1    CIRCULAR SHAPED  0.02245   579.00    0.0130    4.000  0.0    0.0      1.0   12.566   215.805   233.069     YES
 180    1    CIRCULAR SHAPED  0.02333   249.00    0.0130    3.000  0.0    0.0      1.0    7.069   102.150   110.322     YES
 181    1    CIRCULAR SHAPED  0.02690   171.00    0.0130    3.000  0.0    0.0      1.0    7.069   109.688   118.463     YES
 182    1    CIRCULAR SHAPED  0.02172   238.00    0.0130    3.000  0.0    0.0      1.0    7.069    98.563   106.448     YES
 183    1   *CIRCULAR SHAPED  0.04098   255.00    0.0130    1.250  0.0    0.0      1.0    1.227    13.112    14.161     YES
 189    1    CIRCULAR SHAPED  0.03735   340.00    0.0130    1.500  0.0    0.0      1.0    1.767    20.355    21.984     YES
 191    1    CIRCULAR SHAPED  0.02533   210.00    0.0130    1.500  0.0    0.0      1.0    1.767    16.763    18.104     YES
 193    1    CIRCULAR SHAPED  0.02546   205.00    0.0130    1.000  0.0    0.0      1.0    0.785     5.700     6.156     NO
 197    1    CIRCULAR SHAPED  0.05306   422.00    0.0130    2.000  0.0    0.0      1.0    3.142    52.250    56.430     YES,
 198    1   *CIRCULAR SHAPED  0.01059   340.00    0.0130    1.250  0.0    0.0      1.0    1.227     6.666     7.199     NO
 199    1    CIRCULAR SHAPED  0.02000   250.00    0.0130    1.500  0.0    0.0      1.0    1.767    14.895    16.087     NO
```

Table 47.
FINAL CONDUIT DIMENSIONS AND CAPACITIES.

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SECTION VIII

REFERENCES

Agnew, R., and C. A. Hansen. "Biological Treatment of Combined Sewer Overflows at Kenosha, Wisconsin," Envirex, Inc. Milwaukee, Wisconsin, 1973.

American Public Works Association. "Water Pollution Aspects of Urban Runoff," Federal Water Pollution Control Administration Contract WP-20-15, 1969.

American Society of Civil Engineers. *Design and Construction of Sanitary and Storm Sewers,* ASCE Manuals and Reports on Engineering Practice No. 37, 1969.

American Soceity of Civil Engineers. *Design and Construction of Sanitary and Storm Sewers*, Manual of Engineering Practice No. 37, 1960

APWA Research Foundation. "The Swirl Concentrator as a Combined Sewer Overflow Regulator Facility," Office of Research and Monitoring, US EPA, EPA-R2-72-008, 1972.

AVCO Economic System Corp. "Storm Water Pollution from Urban Land Activity," EPA Report No. 11034 FKL 07/70, Washington, DC, 1970.

Brandt, G. H. *et al.* "An Economic Analysis of Erosion and Sediment Control Methods for Watersheds Undergoing Urbanization," Final Report, OWRR Contract No. 14-31-001-3392, Dow Chemical Co., Midland, Michigan, 1972.

Carlisle, A., A. H. F. Brown, And E. J. White. "Litter Fall Leaf Production and the Effects of Defoliation by *Tortrix viridanal* in a Sissile Oak (*Quercus Petrala*) Woodland," *Journal of Ecology*, Vol. 54, 1966.

Chandler, R. F. "The Amount and Mineral Nutrient Content of Freshly Fallen Leaf Litter in the Hardwood Forests of Central New York," *Journal of the American Society of Agronomy*, Vol. 33, No. 10, 1941.

Chandler, R. F. "The Amount and Mineral Nutrient Content of Freshly Fallen Needle Litter of Some Northeastern Conifers," *Proceedings of Soil Science of America,* Vol. 8, 1944.

Charnes, A., and W. W. Cooper. *Management Models and Industrial Applications of Linear Programming,* Vol. I, John Wiley and Sons, Inc., New York, 1961.

Chase, E. S., and A. F. Ferullo. "Oxygen Demand of Leaves in Water," *Water and Sewage Works,* Vol. 105, No. 5, 1958.

Chow, V. T., Ed., *Handbook of Applied Hydrology,* McGraw-Hill Book Co., New York, 1964.

Clark, J. W., W. Viessman, and M. J. Hammer. *Water Supply and Pollution Control,* International Textbook Company, Scranton, Pa., 1971.

Coile, T. S. "Composition of the Leaf Litter of Forest Trees," *J. Elisha Mitchell Science Society,* Vol. 52, 1936.

Corps of Engineers, Everglades Gaging Program, Everglades Area, Progress Report, Jacksonville District, Report No. 6, 1956.

Dantzig, G. B., and P. Wolfe. "Decomposition Principle for Linear Programs," *Operations Research,* Vol. 8, No. 1, 1960.

Daubenmire, R. "Nutrient Content of Leaf Litter of Trees in the Northern Rocky Mountains," *Ecology,* Vol. 34, 1953.

Drainage Criteria Manual, Denver Regional Council of Governments, Vol. 1, 1969.

Eckenfelder, W. W., and J. McCabe. *Biological Treatment of Sewage and Industrial Waste,* Reinhold Publishing Co., New York, 1956.

Environmental Protection Agency. "Environmental Impact of Highway De-icing, EPA Report No. 11040GKK06/71, Edison Water Quality Lab., Edison, NJ, 1970.

Environmental Protection Agency. "Storm Water Management Model," Water Pollution Control Research Series, Washington, DC, 1971:

a. "Volume I, Final Report," Rept. No. 11024DOC07/71
b. "Volume II, Verification and Testing," Rept. No. 11024DOC08/71
c. "Volume III, User's Manual," Rept. No. 11024DOC09/71
d. "Volume IV, Program Listing," Rept. No. 11024DOC10/71

Field R., and E. J. Struzeski, Jr. "Management and Control of Combined Sewer Overflows," *J. Water Pollution Control Federation,* Vol. 44, No. 7, 1972.

Franklin Institute Research Laboratories. "Investigation of Porous Pavements for Urban Runoff Control," EPA Office of Research Monitoring, Rept. No. 11034DUY03/72, Washington, DC, 1972.

Flaxman, E. M. "Predicting Sediment Yield in Western United States," *Journal of the Hydraulics Division, ASCE,* Hy 12, 1972.

"Florida Field Crop Variety Report - 1971," Whitty, E. B. ed., Agronomy Mimeo Report AG 72-51, Institute of Food and Agricultural Sciences, University of Florida, Gainesville, 1972.

Glover, G. E. "High Rate Disinfection of Combined Sewer Overflow," Cochrane Division Crane Co., 1972.

Glover, G. E. "Application of Microstraining to Combined Sewer Overflow," Cochrane Division Crane Co., 1972.

Gameson, A.L., and R. N. Davidson. "Storm Water Investigations at Northhampton," Institute of Sewage Purification, Conference Paper No. 5, Annual Conference, Leandudno, England, 1962.

Gorham, E., and J. R. Bray. "Litter Production in Forests of the World," *Advances in Ecological Research,* Vol. 2, 1964.

Gunn, R., and G. O. Kinzer. "The Terminal Velocity of Fall for Water Droplets," *Journal of Meteorology,* Vol. 6, pp. 243-248, 1949.

Haupt, H. "The Effect of the Decomposition of Natural Organic Contaminating Matter on the Properties of Surface Waters," *Wasser,* Vol. 7, 1933.

Hayes, A. J. "Studies on the Decomposition of Coniferous Leaf Litter. I. Physical and Chemical Changes," *Journal of Soil Science,* Vol. 16, No. 1, 1965.

Heaney, J. P., and L. C. Doughty. "Optimal Atmospheric Emission Control in the Wood Pulp Industry," *Atmospheric Environment,* Vol. 6, 1972.

Heaney, J. P., and W. C. Huber. "Simulation of the Impact of Selected Water Management Alternatives on Stages in Water Conservation Area 3A," Final Report to Central and Southern Florida Flood Control District, 1972.

Heaney, J. P., and R. H. Sullivan. "Source Control of Urban Water Pollution," *Journal of Water Pollution Control Federation,* Vol. 43, No. 4, 1971.

Heaney, J. P. "Mathematical Programming Analysis of Regional Water-Resource Systems," Proc. National Symposium on the Analysis of Water Resource Systems, AWRA, Denver, Colorado, 1968.

Heaney, J.P. "Mathematical Programming Model for Long-Range River Basin Planning with Emphasis on the Colorado River Basin," Ph.D. Dissertation, Northwestern University, Evanston, Illinois 1968.

Henderson, F. M. *Open Channel Flow*, MacMillan, New York, 1966.

Herzog, H. W. "The Air Diffusion Model as an Urban Planning Tool," *Socio-Economic Planning Sciences*, Vol. 3, 1969.

Heyward, F., and R. M. Barnette. "Field Characteristics and Partial Chemical Analyses of the Humus Layer of Longleaf Pine Forest Soils," Bulletin of Florida Agricultural Experiment Station, Vol. 302, 1936.

Highway Research Board. *Parking Principles,* Special Report No. 125, NAS, Washington, DC, 1971.

Hydrotechnic Corporation. "High Rate Filtration of Combined Sewer Overflows," US EPA, Rept. No. 11023EY104/72, Washington, DC, 1972.

Kawahara, Teruhiko. "The Return of Nutrients with Litter in the Forest Ecosystems. II. The Amount of Organic Matter and Nutrients," *Journal of Japanese Forest Society,* Vol. 53, 1972.

Kirshen, P. H., D. H. Marks, and J. C. Schaake. "Mathematical Model for Screening Storm Water Control Alternatives," Ralph M. Parsons Lab Rept. No. 157, Dept. of Civil Engineering, MIT, Cambridge, Massachusetts, 1972.

Kucera, C. "Weathering Characteristics of Deciduous Leaf Litter," *Ecology*, Vol. 40, No. 3, 1959.

Laws, J. O., and D. A. Parsons. "The Relation of Rain Drop Size to Intensity," *Trans. Amer. Geophy. Union,* Vol. 24, p. 452-459, 1943

Leopold, L. B. "The Hydrologic Effects of Urban Land Use," Circular 554, US Geological Survey, Washington, DC, 1968.

Leopold, L. B., F. E. Clark, B. B. Hanshaw, and J. R. Balsley, "A Procedure for Evaluating Environmental Impact," Circular No. 645, US Geological Survey, Washington, DC, 1971.

Loehman, E., and A. Whinston. "A New Theory of Pricing and Decision-Making for Public Investment," *The Bell Journal of Economics and Management Science,* Vol. 2, No. 2, 1971.

Luce, R., and H. Raiffa. *Games and Decisions*, John Wiley and Sons, Inc., New York, 1957.

Lutz, H. J., and R. F. Chandler. *Forest Soils*, John Wiley and Sons, Inc., New York, 1946.

Malcolm, R. L., and R. J. McCracken. "Canopy Drip: A Source of Soil Organic Matter for Mobilization of Iron and Aluminum," *Soil Science Society of America*, Proceedings, Vol. 32, No. 6, 1968.

Maryland Water Resources Administration. "Technical Guide to Erosion and Sediment Control Design," Maryland Department of Natural Resources, September 1973.

Mayer-Krapoll, H. "The Use of Commercial Fertilizers--Particularly Nitrogen--in Forestry," Translated and Published by Nitrogen Division, Allied Chemical and Dye Corp., New York, 1956.

McGriff, Jr., E. C. "The Effects of Urbanization on Water Quality," *Journal of Environmental Quality*, Vol. I, No. 1, 1972.

Melin, E. "Biological Decomposition of Some Types of Litter from North American Forests," *Ecology*, Vol. II, No. 1, 1930.

Metcalf, L., and H. P. Eddy. *American Sewerage Practice*, Vol. III, McGraw-Hill Book Company, Inc., 1935.

Meyer, L. D., and L. A. Kramer. "Erosion Equations Predict Land Slope Developments," *Agricultural Engineer*, Vol. 50, 1969.

Moir, W. H., and H. Grier. "Weight and Nitrogen, Phosphorus, Pottasium, and Calcium Content of Forest Floor Humus of Lodgepole Pine Stands in Colorado," *Soil Science Society of America*, Proceedings, Vol. 33, 1969.

Morrison, J. R. "Variation with Crown Postion and Leaf Age in Content of Seven Element in Leaves of *Pinus Banksiana*," *Canadian Journal of Forest Research*, Vol. 2, 1973.

Mutchler, C. K., and R. A. Young. "Soil Detachment by Raindrops." Presented at Sediment Yield Workshop, USDA Sedimentation Laboratory Oxford, Mississippi, Nov. 28-30, 1972.

National Association of Home Builders Research Foundation, Inc. *A Manual for Residential Street Development Standards, Interim Copy*, Rockville, Maryland, 1973.

O'Conner, D. J., and W. E. Dobbins. "Mechanism of Reaeration in Natural Streams," *Trans. ASCE*, Vol. 123, 1958.

Owen, G. *Game Theory*, W. B. Saunders Co., Philadelphia, Pennsylvania, 1968.

Paquette, R. J., N. Ashford, and P. H. Wright. *Transportation Engineering*, Ronald Press Co., New York, 1972.

Perez, A. I. "A Water Quality Model for a Conjunctive Surface-Groundwater System," Ph.D. Thesis, Department of Environmental Engineering Sciences, University of Florida, Gainesville, 1972.

Perez, A. I., W. C. Huber, J. P. Heaney, and E. E. Pyatt. "Water Quality Model for Conjunctive Surface-Groundwater System," *Water Resources Bulletin*, Vol. 8, No. 5, 1972.

Poertner, H. G. "Better Storm Drainage Facilities - At Lower Cost," *Civil Engineering*, ASCE, October 1973.

Ports, M. A. "Use of the Universal Soil Loss Equation as a Design Standard," ASCE Water Resources Engineering Meeting, Washington, DC, 1973.

Pyatt, E. E., J. P. Heaney, G. R. Grantham, and B. J. Carter. "A Model for Quantifying Flow Augmentation Benefits," Final Report to FWPCA, 1969.

Rapoport, A. *N-Person Game Theory*, University of Michigan Press, 1970.

Rex Chainbelt, Milwaulkee, Wisconsin. "Screening/Flotation Treatment of Combined Sewer Overflows," US EPA, Rept. No. 11020FPC01/72, Washington, DC, 1972.

Rohrer Associates, Inc. "Underwater Storage of Combined Sewer Overflows," US EPA, Rept. No 11022ECU09/71, Washington, DC, 1971.

Ruelke, O. C., and G. M. Prine. "Performance of Six Hybrid Bermudagrass, Pangola Digitgrass, and Pensacola Bahiagrass at Three Fertility Levels in North Central Florida," *Soil and Crop Science Society of Florida*, Proceedings, Vol. 31, 1971.

Ruelke, O. C., and G. M. Prine. "Preliminary Evaluation of Yield and Protein Content of Six Hybrid Bermudagrass, Pensacola Bahiagrass, and Pangolagrass under Three Fertilization Regimes in North Central Florida," *Soil and Crop Science Society of Florida*, Proceedings, Vol. 28, 1968.

Sartor, J. D., and B. B. Gail. "Water Pollution Aspects of Street Surface Contaminants," Office of Research and Monitoring, EPA, Rept. No. EPA-R2-72-081, Washington, DC, 1972.

Slack, K. V., and H. R. Feltz. "Tree Leaf Control on Low Flow Water Quality in a Small Virginia Stream," *Environmental Science and Technology*, Vol. 2, 1968.

Slack, K. V. "Effect of Tree Leaves on Water Quality in Cacapon River, West Virginia," *U. S. Geological Survey Professional Papers*, No. 475-D, 1964.

Smith, R. "Preliminary Design of Simulation of Conventional Waste Water Renovation Systems Using the Digital Computer," U.S. Dept. of the Interior, FWPCA, 1968.

Soil Conservation Service. *SCS National Engineering Handbook, Section 4--Hydrology*, USDA, Washington, DC, 1972.

Sorenson, S. W. "A Mathematical Theory of Coalitions and Competition in Resource Development," Ph.D. Dissertation, University of Texas at Austin, 1972.

Sorman, V. A., and J. R. Wallace. "Digital Simulation of Thunderstorm Rainfall," Environmental Resources Center Rept. ERC-0972, Georgia Institute of Technology, Atlanta, 1972.

Stall, J. B., and M. L. Terstriep. "Storm Sewer Design--An Evaluation of the RRL Method," EPA, Office of Research and Monitoring, Rept. No. EPA-R2-72-068, Washington, DC, 1972.

TRW. *Air Quality Display Model*, USDHEW, PHS. Pub. No. PB 189-194, 1969.

Urban Land Institute. *The Community Builders Handbook*, Washington, DC, 1968.

White, E. M. "Water-Leachable Nutrients from Frozen or Dried Prairie Vegetation," *Journal of Environmental Quality*, Vol. 2, No. 1, 1973.

Williams, J. R., and H. D. Berndt. "Sediment Yield Computed with Universal Equation," *Journal of Hydraulics Division*,HY 12, ASCE, December 1972.

Wilson, A. L. "An Atmospheric Diffusion Model for Jacksonville, Florida," M.S. Thesis, Department of Environmental Engineering Sciences, University of Florida, Gainesville, 1971.

Wischmeier, W. H., C. B. Johnson, and B. U. Cross. "A Soil Erodibility Nomograph for Farmland and Construction Sites," *Journal of Soils and Water Conservation*, Vol. 26, 1971.

Wischmeier, W. H., and D. D. Smith. "Predicting Rainfall-Erosion Losses from Cropland East of the Rocky Mountains," Agr. Handbook 282, US Dept. of Agriculture, Washington, DC, 1965.

Wischmeier, W. H., and D. D. Smith. "Rainfall Energy and Its Relationship to Soil Loss," *Transactions, American Geophysical Union*, Vol. 39, No. 2, 1958.